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# Storm Water Management Model

## *Volume I—Final Report*

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The Water Pollution Control Research Reports describe the results and progress in the control and abatement of pollution of our Nation's waters. They provide a central source of information on the research, development and demonstration activities of the Water Quality Office of the Environmental Protection Agency, through in-house research and grants and contracts with the Federal, State and local agencies, research institutions, and industrial organizations.

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To be continued on inside back cover...

# STORM WATER MANAGEMENT MODEL

## Volume I - Final Report

by

Metcalf & Eddy, Inc., Palo Alto, California
University of Florida, Gainesville, Florida
Water Resources Engineers, Inc., Walnut Creek, California

for the

ENVIRONMENTAL PROTECTION AGENCY

Contract No. 14-12-501 Project No. 11024EBI
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# ABSTRACT

A comprehensive mathematical model, capable of representing urban storm water runoff, has been developed to assist administrators and engineers in the planning, evaluation, and management of overflow abatement alternatives.

Hydrographs and pollutographs (time varying quality concentrations or mass values) were generated for real storm events and systems from points of origin in real time sequence to points of disposal (including travel in receiving waters) with user options for intermediate storage and/or treatment facilities. Both combined and separate sewerage systems may be evaluated. Internal cost routines and receiving water quality output assisted in direct cost-benefit analysis of alternate programs of water quality enhancement.

Demonstration and verification runs on selected catchments, varying in size from 180 to 5,400 acres, in four U.S. cities (approximately 20 storm events, total) were used to test and debug the model. The amount of pollutants released varied significantly with the real time occurrence, runoff intensity duration, pre-storm history, land use, and maintenance. Storage-treatment combinations offered best cost-effectiveness ratios.

A user's manual and complete program listing were prepared.

This report was submitted in fulfillment of Projects 11024 EBI, DOC, and EBJ under Contracts 14-12-501, 502, and 503 under the sponsorship of the Environmental Protection Agency.

The titles and identifying numbers of the final report volumes are:

| Title | EPA Report No. |
|---|---|
| STORM WATER MANAGEMENT MODEL<br>Volume I - Final Report | 11024 DOC 07/71 |
| STORM WATER MANAGEMENT MODEL<br>Volume II - Verification and Testing | 11024 DOC 08/71 |
| STORM WATER MANAGEMENT MODEL<br>Volume III - User's Manual | 11024 DOC 09/71 |
| STORM WATER MANAGEMENT MODEL<br>Volume IV - Program Listing | 11024 DOC 10/71 |

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# SECTION 1

# CONCLUSIONS AND RECOMMENDATIONS

## CONCLUSIONS

Under the sponsorship of the Environmental Protection Agency a consortium of contractors--Metcalf & Eddy, Inc., the University of Florida, and Water Resources Engineers, Inc.--has developed a comprehensive mathematical model capable of representing urban storm water runoff and combined sewer overflow phenomena. Correctional devices in the form of user selected options for storage and/or treatment are provided with associated estimates of cost. Effectiveness is portrayed by computed treatment efficiencies and modeled changes in receiving water quality.

### Capabilities of the Model

1. The Storm Water Management Model accepts any rainfall hyetograph or multiple hyetographs (where several local rain gages are recording the same storm event) and produces a runoff hydrograph for each modeled watershed.

2. Continuous runoff quality graphs, called pollutographs, are computed on the basis of the volume of storm runoff and on antecedent conditions which include rainfall history, street sweeping data, land use, and related data.

3. Hydrographs and pollutographs are computed for dry weather flows (in the case of combined systems) with daily and hourly variations and for infiltration.

4. Real time flow routing is accomplished through a simulation of the physical collection system allowing for variations in size, length, slope, and cross-section configuration in the case of conduits and locations and design parameters for non-conduits.

5. User options for storage facilities may be internal (in-system) or external (located at the overflow point). Options for storage are:

   a. Surface storage
   b. Intrasystem storage
   c. Flow rerouting
   d. Underground storage
   e. Underwater storage.

6. User options for treatment may be called in conjunction with or independent of external storage. The following treatment options and acceptable combinations of these options are provided:

   a. Mechanically cleaned bar racks
   b. Fine screens
   c. Sedimentation
   d. Dissolved air flotation
   e. Microstrainers
   f. High rate filters
   g. Effluent screens
   h. Chlorination.

7. Receiving water effects in both flow and quality are computed for simulated lakes, rivers, or estuaries as applicable.

8. Capital costs and operation and maintenance costs are computed for the options selected to permit the development of approximate cost-effectiveness curves which can be used for decision making.

Applications of the Model

1. The Storm Water Management Model is written in FORTRAN programming language with a minimum of machine-dependent features. To date the program has been tested on four independent computer hardware systems.

2. The computer hardware system should be the equivalent of the IBM 360/65 with peripheral storage devices and a usable core capacity of not less than 350K bytes.

3. The Model is developed on a general basis so it may be applied to any municipality by changing only the input data. The approximate size range of drainage basins over which the Model is applicable is from 10 acres to 5,000 acres.

4. A "User's Manual," Volume III, is furnished describing the main computer program, all subroutines, data input and output, and complete instructions on applications to individual systems.

5. The user should be knowledgeable in FORTRAN programming, operating systems interfacing, and the engineering aspects of the real systems.

6. Data requirements are common to engineering design and analysis and are mainly descriptive of the real systems.

## Limitations of the Model

1. Data analyses and interpretation are essential.

2. In its present form only one drainage basin is studied during a computational run. Thus, the combined effects of multiple coincident discharges into a single receiving water body are not directly evaluated. However, the multiple discharge effect can be simulated by executing a series of runs (one per basin up to a maximum of 20), creating an output master file of the discharge solutions, and then applying this master file as input to the Receiving Water Model.

3. Quality and perhaps quantity variations due to geographical or seasonal differences are inadequately documented due to the sparsity of equivalent real system data.

4. Costs and treatment data in many instances are based on advanced processes which have not been in operation beyond the pilot plant or limited demonstration plant stage. The cost-effectiveness of the treatment alternatives should then be viewed in the light of this limitation.

5. The program, being completely new and of considerable size, will require much use and continuous modification to fulfill most effectively its objective of providing a decision-making tool.

6. The Model, as structured, does not optimize to a specific solution, but rather provides a comprehensive analysis of user selected alternatives.

7. The present Model simulates individual storm events which may or may not represent the random occurrence and probabilistic nature of the real

hydrologic phenomena. However, subject to fund limitations, any number of independent storms may be run separately to support such analyses.

## Results of Demonstration-Verification Studies

1. The real time occurrence of the storm event (hour of the day) significantly affects the quality of combined sewer overflows.

2. The intensity-duration relationships of the runoff (i.e., the shape of the hydrograph), as well as the total runoff amount, must be considered in evaluating combined sewer overflow abatement alternatives.

3. Idealized "design storms" (i.e., the traditional 2-year, 5-year, and 10-year storm) may give a poor representation of the quality characteristics of overflows by failing to represent the time of occurrence and shape of the hydrograph.

4. The use of real storm data "design event(s)," together with surface and system simulation in the EPA Storm Water Management Model, offer a means of significantly improved characterization of particular drainage basins.

5. The first flush effect is a function of the antecedent dry weather flow, the transport system geometry, and the design event. The dry weather flow provides the source, the transport system determines location and rate of solids accumulation, and the design event determines the rate of uptake and travel time to the overflow point.

6. Quality concentrations alone are not adequate indicators of the pollution potential of combined sewer overflows and storm water discharges

because of the high variation in flow rates during design events. Mass (flow times concentration) discharge rates are generally more significant for the long term effects, and concentrations are perhaps more significant for the short term shock effects.

7. The areal distribution of rainfall can significantly influence runoff quality.

8. For the demonstration situations modeled, the combination of storage followed by high rate treatment offered the best cost-effectiveness ratios. Since generally only single storms were modeled, the study was not pursued to determine the most favorable ratios between storage and treatment capacities.

9. The dynamic modeling of the receiving waters permits the evaluation of the transient load problem as well as the pollutant effects averaged out to steady state conditions.

## RECOMMENDATIONS

1. It is recommended, above all else, that the Storm Water Management Model be put to work on real systems with real problems. The use, evaluation, and continuous updating of the Model are essential to its credibility and effectiveness.

2. It is recommended that the Model be used in the research and development of representative storm events based upon years of rainfall record to provide an economical base for evaluating alternate courses of corrective action.

3. It is recommended that the Model be further developed to encompass the effects of multiple coincident discharges to common receiving waters. Storm and combined sewer, treatment plant, tributary stream, and other significant discharges should be considered. For economy, a correlation of overflows from non-modeled watersheds with a limited number of modeled representative areas should be established as a basis for extrapolation. Such development would be directed toward a truer measure of water quality improvement accompanying specific steps of corrective action.

4. It is recommended that data gathering efforts be continued to improve the definition of waste sources and the removals induced by urban storm water runoff, including the extension to additional quality constituents.

5. It is recommended that the Model be used in conjunction with Demonstration Projects for advanced combined sewer overflow waste treatment processes to assist in the evaluation and optimization of their design and operation.

6. Finally, it is recommended that the Storm Water Management Model be used as a control guide in setting up procedures for the retention, redistribution, treatment, and release of excess flows resulting from storm events at the time of occurrence.

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## INTRODUCTION

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## SECTION 2

## INTRODUCTION

An expenditure of billions of dollars is expected in the next decade in the United States to combat the degradation of streams and other bodies of water by pollutants discharged through combined sewer overflows or separate storm sewer discharges. The Environmental Protection Agency (EPA) is therefore directing or assisting in multiple research and development programs and investigations to identify, control, and correct known problems relating to these storm occurrences.

One of these research programs, the development and testing of a comprehensive simulation model to represent completely storm event phenomena in urban areas, is the subject of this report.

### PROJECT CONCEPTION

In April 1967, Metcalf & Eddy, Inc., (M&E) submitted a proposal envisioning the use of systems engineering techniques to identify storm and sanitary flows and characteristics, conveyance systems, and the pollutional effects of storm water upon the environment. This and earlier proposals, related in subject matter, by the University of Florida (UF) and Water Resources Engineers, Inc., (WRE), upon review and evaluation by the EPA's Storm and Combined Sewer Pollution Control Branch, led to the concept of the comprehensive mathematical model. As evolved this model would represent

> "...urban storm water runoff phenomena, both quantity and quality, from the onset of precipitation on the basin, through collection, conveyance, storage, and treatment systems, to points downstream from outfalls which are significantly affected by storm discharges."

The goal was to provide a fundamental tool for administrators and designers to assist them in the management and allocation of resources towards optimal storm water quality control. Because the model was to be universal in its application and all-encompassing in its detail, and because the duration of the project was restricted to 18 months, a cooperative effort drawing upon the talents and resources of all three contractors and the EPA was decided upon. A demonstration-verification phase was incorporated into the work to insure the validity of the results and the utility of its application.

## STATEMENT OF WORK

### Objective A, Intent

The first objective, model development, involved several component tasks. The goal of each task involving simulation (mathematical modeling) was to provide a computer program subroutine which could be directly applied to the comprehensive model. All subroutines were designed to be dynamic, reflecting time and space variations. All previous data in each task area were examined, evaluated, and used if suitable. No sampling was performed. All subroutines were developed on a general basis so they could be applied to any municipality by changing only the input data. A sub-basin, multi-level branched system concept was used. The model was designed to be applicable over drainage basins from 10 to 5,000 acres, and to be operational on the Department of the Interior computation facilities.

### Objective A, Tasks and Assignments

Table 2-1 summarizes the component task categories and assignments.

Table 2-1. COMPONENT TASK CATEGORIES AND ASSIGNMENTS

| Tasks | Assignment |
|---|---|
| MANAGEMENT | |
| 1. Be responsible for overall coordination and management | M&E |
| 2. Be responsible for final development of the comprehensive model | M&E |
| 3. Participate in coordination meetings and provide consultation and guidance | UF - WRE |
| QUANTITY (HYDROLOGIC) SUBROUTINES | |
| 1. Develop urban runoff model | WRE |
| 2. Develop simulation model for dry weather sewage flows | UF |
| 3. Develop infiltration model | UF |
| 4. Develop transport model, simulating the branched collection system and main sewer | UF |
| 5. Develop storage simulation model | M&E |
| 6. Develop receiving water flow model for creek, river, estuary or lake | WRE |
| QUALITY SUBROUTINES to be integrated with associated quantity (hydrologic) subroutines | |
| 1. Develop runoff quality model | M&E |
| 2. Develop dry weather flow quality model | M&E |
| 3. Develop decay model quality routing in transport model | UF |
| 4. Develop receiving water quality model | WRE |
| 5. Develop treatment simulation model | M&E |
| ECONOMIC DATA | |
| 1. Develop cost model for approximating cost-effectiveness curves | M&E |

Under Objective A, M&E provided overall direction and coordination of the project and developed subroutines representing the quality aspects of storm water runoff and DWF; all storage and treatment options; and a cost model representing these options. UF developed subroutines representing DWF quantity and infiltration; the transport simulation, both quantity and quality; and the decay of constituents in transport. WRE developed subroutines representing urban storm water runoff quantity phenomena; and all receiving waters, both quantity and quality. EPA furnished available process and cost data on the storage and treatment options.

Objective B, Intent

The second objective, demonstration-verification, was to test the validity of the Model by applying it to several storm and combined sewer drainage basins. Possible solutions to existing problems were proposed by manipulating flow control and treatment alternatives in the Model. The apparent best solutions based upon cost-effectiveness were indicated, disregarding sociopolitical factors.

Objective B, Assignments

M&E continued coordination of the overall project, conducted all computer runs of the comprehensive Model and associated data reduction to proper card/tape input form, and was responsible for demonstrating the operation of the program on the EPA computer. WRE and UF provided necessary analysis of computer output and refined and debugged their respective programs. EPA made the initial contacts with the study municipalities and provided the majority of the raw data.

## METHOD OF APPROACH

The activity chart used for project management and control is shown in Figure 2-1. An initial meeting and five subsequent quarterly meetings, each of two days' duration, were jointly attended by all contractors and the EPA to ensure coordination and direct transfer and critique of information.

The computer language, FORTRAN IV, and primary hardware, IBM 360/65, were selected as most compatible with Environmental Protection Agency requirements and universal acceptance. To ensure high flexibility in the ultimate program usage, some programs were developed and all were tested on a UNIVAC 1108 as well as on the primary system.

The "triumvirate" approach enabled simultaneous starts in three prime program areas--runoff, quality, and transport--and working routines were well in hand by the six-month target date, permitting the first comprehensive Model run at that time.

Use of real systems data was sought and integrated into each subroutine's development, although in many instances the available data were far too sparse and/or lacking in detail to be of maximum benefit. Contacts with demonstration municipalities in the third and fourth quarters enabled a prompt start on verification activities upon completion of the first year of work and final completion at the end of 18 months.

## PRESENTATION FORMAT

The project report is divided into four volumes. This volume, the "Final Report," contains the background, justifications, judgments, and assumptions

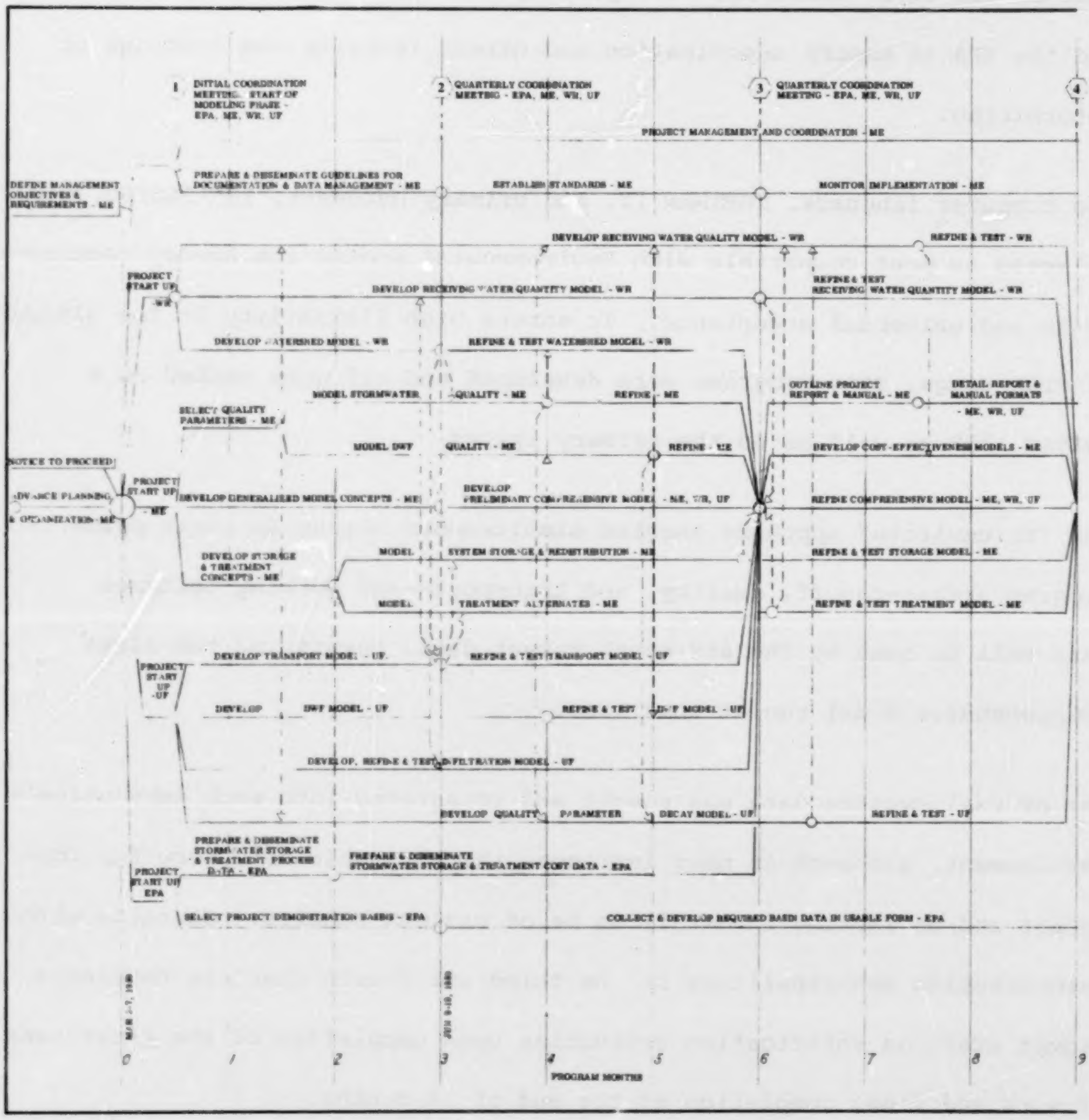
1
INITIAL COORDINATION MEETING. START OF MODELING PHASE - EPA, ME, WR, UF
2
QUARTERLY COORDINATION MEETING - EPA, ME, WR, UF
3
QUARTERLY COORDINATION MEETING - EPA, ME, WR, UF
4
PROJECT MANAGEMENT AND COORDINATION - ME
DEFINE MANAGEMENT OBJECTIVES & REQUIREMENTS - ME
PREPARE & DISSEMINATE GUIDELINES FOR DOCUMENTATION & DATA MANAGEMENT - ME
ESTABLISH STANDARDS - ME
MONITOR IMPLEMENTATION - ME
DEVELOP RECEIVING WATER QUALITY MODEL - WR
REFINE & TEST - WR
PROJECT START UP - WR
DEVELOP RECEIVING WATER QUANTITY MODEL - WR
REFINE & TEST RECEIVING WATER QUANTITY MODEL - WR
DEVELOP WATERSHED MODEL - WR
REFINE & TEST WATERSHED MODEL - WR
MODEL STORMWATER QUALITY - ME
REFINE - ME
OUTLINE PROJECT REPORT & MANUAL - ME
DETAIL REPORT & MANUAL FORMATS - ME, WR, UF
SELECT QUALITY PARAMETERS - ME
MODEL DWF QUALITY - ME
REFINE - ME
DEVELOP COST-EFFECTIVENESS MODELS - ME
NOTICE TO PROCEED
ADVANCE PLANNING & ORGANIZATION - ME
PROJECT START UP - ME
DEVELOP GENERALIZED MODEL CONCEPTS - ME
DEVELOP PRELIMINARY COMPREHENSIVE MODEL - ME, WR, UF
REFINE COMPREHENSIVE MODEL - ME, WR, UF
DEVELOP STORAGE & TREATMENT CONCEPTS - ME
MODEL SYSTEM STORAGE & REDISTRIBUTION - ME
REFINE & TEST STORAGE MODEL - ME
MODEL TREATMENT ALTERNATES - ME
REFINE & TEST TREATMENT MODEL - ME
DEVELOP TRANSPORT MODEL - UF
REFINE & TEST TRANSPORT MODEL - UF
PROJECT START UP - UF
DEVELOP DWF MODEL - UF
REFINE & TEST DWF MODEL - UF
DEVELOP, REFINE & TEST INFILTRATION MODEL - UF
DEVELOP QUALITY PARAMETER DECAY MODEL - UF
REFINE & TEST - UF
PREPARE & DISSEMINATE STORMWATER STORAGE & TREATMENT PROCESS DATA - EPA
PROJECT START UP EPA
PREPARE & DISSEMINATE STORMWATER STORAGE & TREATMENT COST DATA - EPA
SELECT PROJECT DEMONSTRATION BASINS - EPA
COLLECT & DEVELOP REQUIRED BASIN DATA IN USABLE FORM - EPA
MAR 5-7, 1980
JUN 9-10, 1980
0
1
2
3
4
5
6
7
8
9
PROGRAM MONTHS

Figure 2-1. ACTIVITY CHART

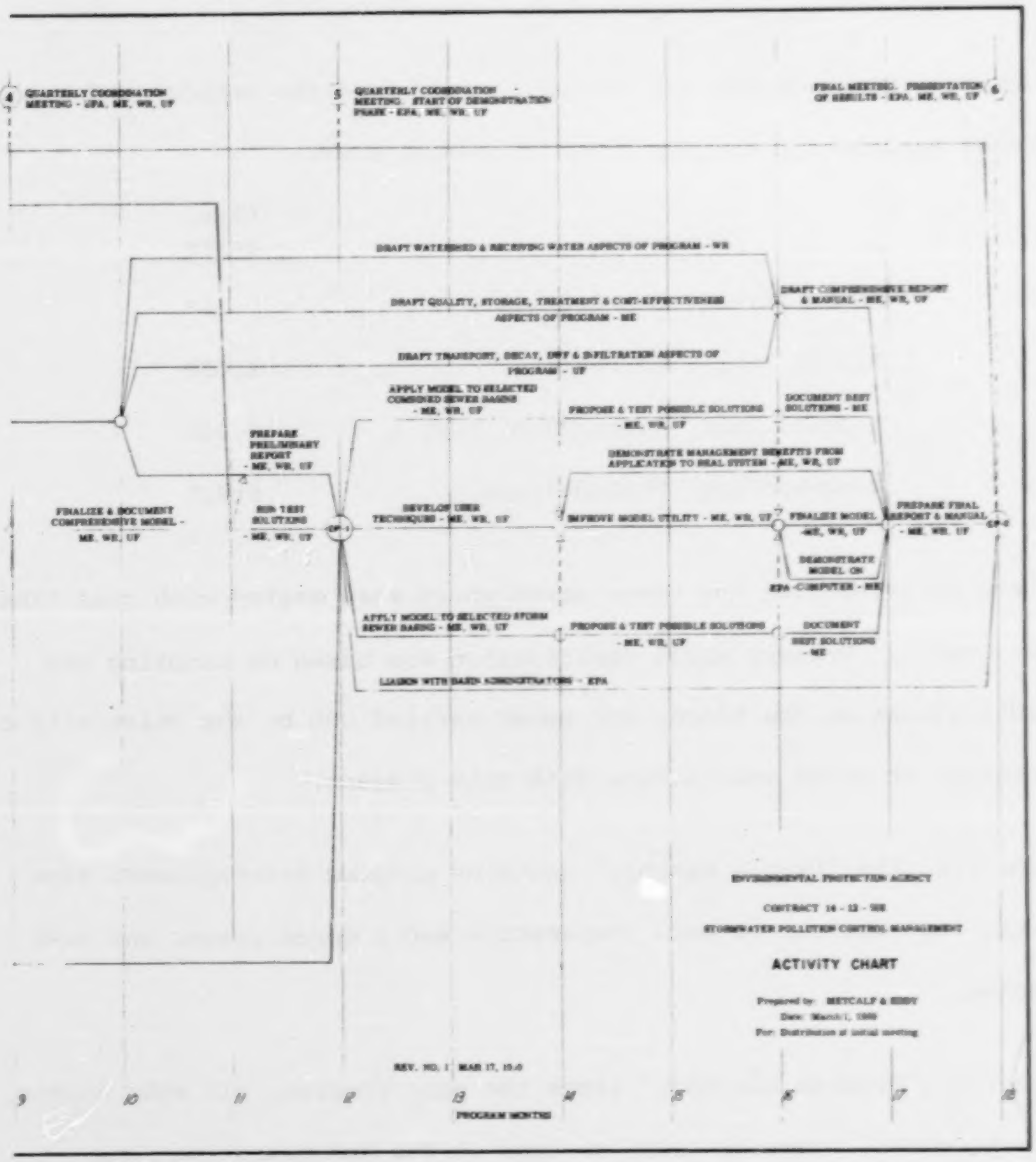
4 QUARTERLY COORDINATION MEETING - EPA, ME, WR, UF
5 QUARTERLY COORDINATION MEETING. START OF DEMONSTRATION PHASE - EPA, ME, WR, UF
FINAL MEETING. PRESENTATION OF RESULTS - EPA, ME, WR, UF 6
DRAFT WATERSHED & RECEIVING WATER ASPECTS OF PROGRAM - WR
DRAFT QUALITY, STORAGE, TREATMENT & COST-EFFECTIVENESS ASPECTS OF PROGRAM - ME
DRAFT TRANSPORT, DECAY, DWF & INFILTRATION ASPECTS OF PROGRAM - UF
DRAFT COMPREHENSIVE REPORT & MANUAL - ME, WR, UF
APPLY MODEL TO SELECTED COMBINED SEWER BASINS - ME, WR, UF
PROPOSE & TEST POSSIBLE SOLUTIONS - ME, WR, UF
DOCUMENT BEST SOLUTIONS - ME
PREPARE PRELIMINARY REPORT - ME, WR, UF
DEMONSTRATE MANAGEMENT BENEFITS FROM APPLICATION TO REAL SYSTEM - ME, WR, UF
FINALIZE & DOCUMENT COMPREHENSIVE MODEL - ME, WR, UF
RUN TEST SOLUTIONS - ME, WR, UF
CP-1
DEVELOP USER TECHNIQUES - ME, WR, UF
IMPROVE MODEL UTILITY - ME, WR, UF
7
FINALIZE MODEL - ME, WR, UF
PREPARE FINAL REPORT & MANUAL - ME, WR, UF
CP-2
DEMONSTRATE MODEL ON EPA COMPUTER - ME
APPLY MODEL TO SELECTED STORM SEWER BASINS - ME, WR, UF
PROPOSE & TEST POSSIBLE SOLUTIONS - ME, WR, UF
DOCUMENT BEST SOLUTIONS - ME
LIAISON WITH BASIN ADMINISTRATORS - EPA
ENVIRONMENTAL PROTECTION AGENCY
CONTRACT 14 - 12 - 501
STORMWATER POLLUTION CONTROL MANAGEMENT
ACTIVITY CHART
Prepared by: METCALF & EDDY
Date: March 1, 1969
For: Distribution at initial meeting
REV. NO. 1 MAR 17, 1970
9
10
11
12
13
14
15
16
17
18
PROGRAM MONTHS

FIGURE 2-1. ACTIVITY CHART (continued)

used in Model development. It further includes descriptions of unsuccessful modeling techniques that were attempted and recommendations for forms of user teams to implement systems analysis techniques most efficiently.

Volume II, "Verification and Testing," describes the methods and results of Model application in four urban catchment areas:

| Location | Size, acres |
|---|---|
| Baker Street, San Francisco | 187 |
| Bloody Run, Cincinnati | 2,600 |
| Kingman Lake, Washington, D.C. | 4,200 |
| Wingohocking, Philadelphia | 5,400 |

All are existing combined sewer areas which were tested with real time storm events. Primary Model verification was based on sampling and investigations on the Bloody Run sewer carried out by the University of Cincinnati in close association with this project.

Volume III, the "User's Manual," contains program descriptions, flow charts, instructions on data preparation and program usage, and test examples.

Volume IV, "Program Listing," lists the main program, all subroutines, and Job Control Language (JCL) as used in the demonstration runs.

# PART I

# COMPREHENSIVE MODEL

# SECTION 3

## MODEL OVERVIEW

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# SECTION 3

## MODEL OVERVIEW

The comprehensive Storm Water Management Model is intended to provide a fundamental tool for modeling and evaluating existing phenomena associated with storm water runoff and combined sewer overflows from urban areas and, through simulation, to indicate system responses to selected means of corrective action.

### THE PROBLEM

Precipitation falling on urban areas becomes contaminated as it enters and passes through or within the human environment (Ref. 1). The first degradation comes from contact with pollutants in the air; the next, through contacts as it passes over ground and building surfaces; and finally, through contacts with residues (depositions from early storms or usage) and/or wastewaters (DWF in combined systems) in the conveyance system and appurtenances. This storm runoff with or without sanitary sewage picked up in transit, will eventually discharge to some receiving body of water where the contaminants will be held for decomposition (nonconservative constituents) or accumulation (conservative constituents), or will be passed on downstream using the receiving waters as a carrier.

How serious is this contamination? Where does it come from? How does it vary from one location to the next, or from one storm event to the next? How do the related phenomena interplay? How can this contamination be controlled? How can alternate schemes of corrective action be compared? How much will they cost? What will be their effects on the receiving waters? These are the basic questions that guided the development of the comprehensive Storm Water Management Model.

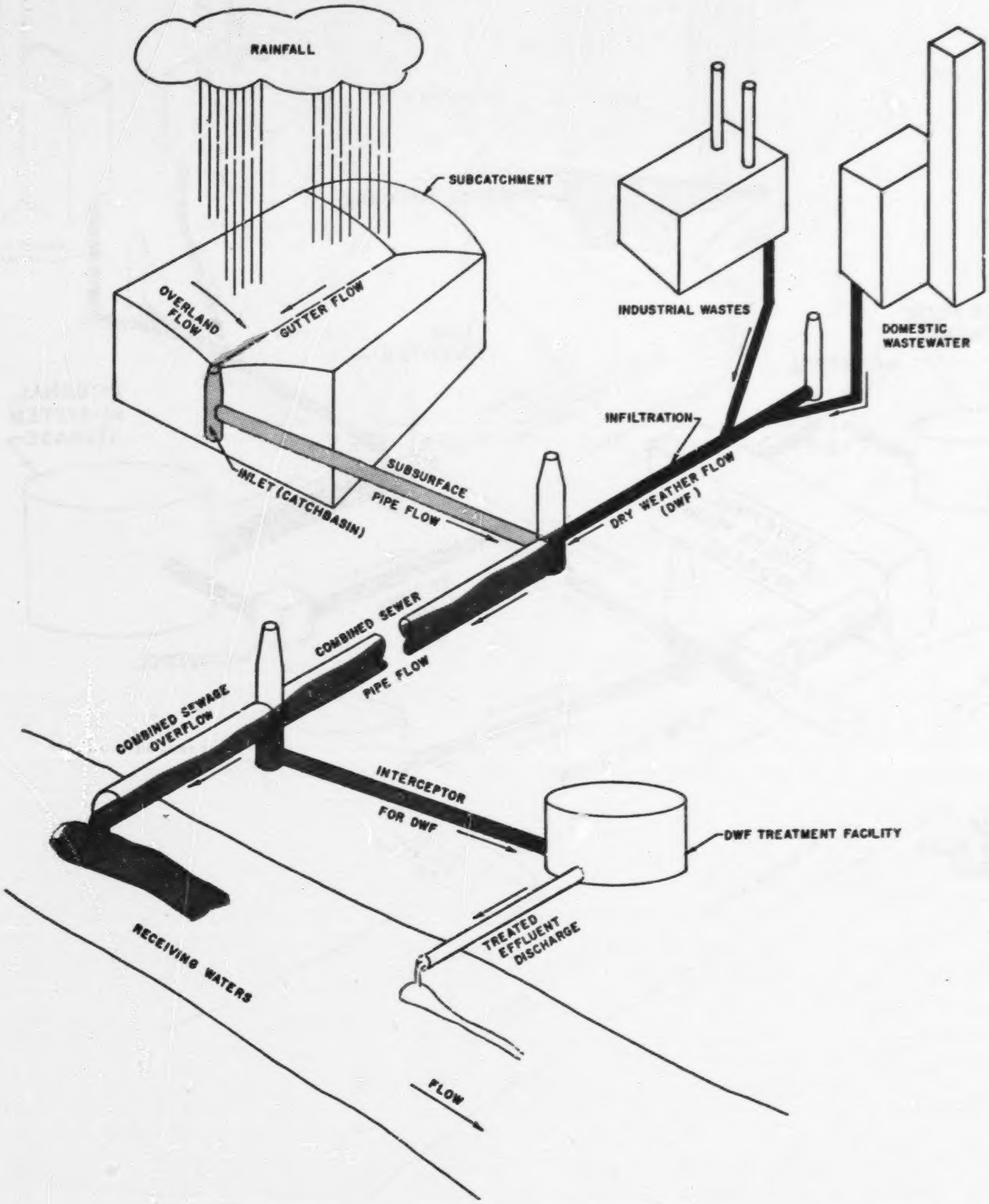
Figure 3-1 shows in pictorial form the origins and movements of flows in typical combined sewerage systems. Figure 3-2 shows some of the storage and treatment systems now under study to attack the problem to protect and/or improve the quality of the receiving waters.

## Dry Weather Flow

DWF is basically the wastewater discharge of a community diluted by such infiltration, groundwater leakage into the conveyance system, as may occur. In volume it is comparatively small, averaging about 100 gal./capita/day, but its pollutant concentrations are high, and if untreated, a menace to the public health or environment. DWF may be conveyed in pipe systems which exclude storm waters (called separate systems) or in systems which handle both DWF and storm water runoff (called combined systems).

## Storm Water Runoff

Storm water runoff is the precipitation, striking a surface during a storm event, which exceeds the infiltration capacity (absorption into the ground) or holding capacity (ponding) of that surface. This excess flow is conveyed through systems of gutters and pipes which may be separate or combined as described above. In volume it may be very large, frequently exceeding the rate of DWF discharge from the same area by 100 times or more, but its pollutant concentrations are generally less concentrated than DWF, although in total mass units they are generally greater if limited to the period of the storm event.


RAINFALL
SUBCATCHMENT
OVERLAND FLOW
GUTTER FLOW
INDUSTRIAL WASTES
DOMESTIC WASTEWATER
INFILTRATION
DRY WEATHER FLOW (DWF)
SUBSURFACE PIPE FLOW
INLET (CATCHBASIN)
COMBINED SEWER PIPE FLOW
COMBINED SEWAGE OVERFLOW
INTERCEPTOR FOR DWF
DWF TREATMENT FACILITY
TREATED EFFLUENT DISCHARGE
RECEIVING WATERS
FLOW

Note: DWF sources actually overlay the subcatchments. They are separated in the figure only to simplify the representation.

Figure 3-1. SCHEMATIC SYSTEM DRAWING RAINFALL THROUGH OVERFLOW

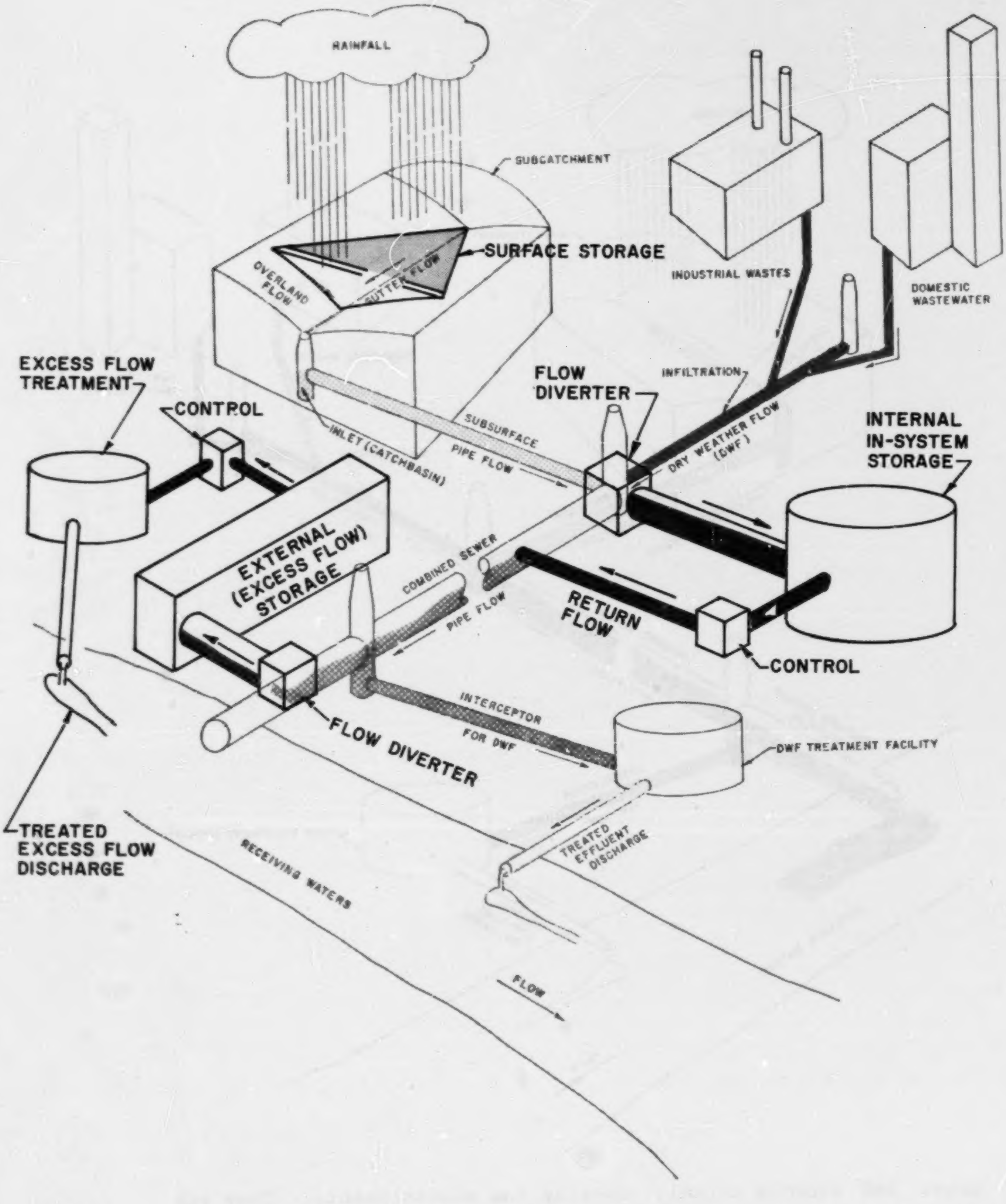

RAINFALL
SUBCATCHMENT
SURFACE STORAGE
OVERLAND FLOW
GUTTER FLOW
INDUSTRIAL WASTES
DOMESTIC WASTEWATER
INFILTRATION
EXCESS FLOW TREATMENT
CONTROL
FLOW DIVERTER
SUBSURFACE PIPE FLOW
INLET (CATCHBASIN)
DRY WEATHER FLOW (DWF)
INTERNAL IN-SYSTEM STORAGE
EXTERNAL (EXCESS FLOW) STORAGE
COMBINED SEWER
PIPE FLOW
RETURN FLOW
CONTROL
INTERCEPTOR FOR DWF
FLOW DIVERTER
DWF TREATMENT FACILITY
TREATED EXCESS FLOW DISCHARGE
RECEIVING WATERS
TREATED EFFLUENT DISCHARGE
FLOW

Figure 3-2. TYPICAL STORAGE-TREATMENT APPLICATIONS TO LIMIT UNTREATED OVERFLOWS

## Combined Sewer Overflows

It is an almost universal practice in the United States to treat DWF prior to its release to receiving waters regardless of the method of conveyance. In combined sewer systems this is accomplished by means of a regulatory device, such as a weir, which in theory, diverts all flows during non-storm periods. During storm events, however, the flows arriving at the regulator may be far in excess of the treatment capacity, and this excess is bypassed directly to the receiving water without treatment. This excess flow constitutes the combined sewage overflow and consists of a mixture of storm runoff, DWF, and such residue as may have been picked up in the conveyance system.

## Corrective Action

The state of the art seeking solutions to combat pollution resulting from storm water runoff and combined sewage overflows centers on (1) improved maintenance practices and housekeeping, (2) storage facilities, and (3) treatment facilities.

Improved maintenance practices tend to clear source material off the streets and out of the conveyance system before the storm event occurs, thereby reducing the contaminants in the storm water flow. They also provide the necessary inspection and maintenance of regulatory and appurtenant devices to insure their proper functioning (Ref. 2).

Storage facilities are used for contaminant and/or flow rate reduction of the storm influenced flow to permit larger quantities to be treated at existing DWF facilities or new excess flow treatment facilities. They may also be used to provide relief for over-extended systems to control or reduce flooding.

Treatment facilities for excess flows may be either on-line or off-line. On-line facilities are exposed to the complete variations of the excess flow, both in flow rate and quality, whereas the off-line facilities may be fed at a controlled rate, the excess being bypassed or temporarily diverted into storage.

Further descriptions and applications of these storage-treatment schemes are discussed later in this volume and in Volume II.

An introduction to the modeling techniques developed to represent the problem phenomena and corrective actions follow.

## THE COMPREHENSIVE MODEL

The comprehensive Storm Water Management Model uses a high speed digital computer to simulate real storm events on the basis of rainfall (hyetograph) inputs and system (catchment, conveyance, storage/treatment, and receiving water) characterization to predict outcomes in the form of quantity and quality values.

The simulation technique--that is, the representation of the physical systems identifiable within the Model--was selected because it permits relatively easy interpretation and because it permits the location of remedial devices (such as a storage tank or relief lines) and/or denotes localized problems (such as flooding) at a great number of points in the physical system.

Since the program objectives are particularly directed toward complete time and spatial effects, as opposed to simple maxima (such as the rational formula approach) or only gross effects (such as total pounds of pollutant discharged in a given storm), it is considered essential to work with continuous curves (magnitude versus time), referred to as hydrographs and "pollutographs." The units selected for quality representation, pounds per minute, identify the mass releases as these portray both the volume and the concentration of the release in a single term. Concentrations are also printed out within the program for comparisons with measured data.

An overview of the Model structure is shown in Figure 3-3. The actual programming structure is discussed in Section 4. In simplest terms the program is built up as follows:

1. The input sources:

   RUNOFF generates surface runoff based on an arbitrary rainfall hyetograph, antecedent conditions, land use, and topography.

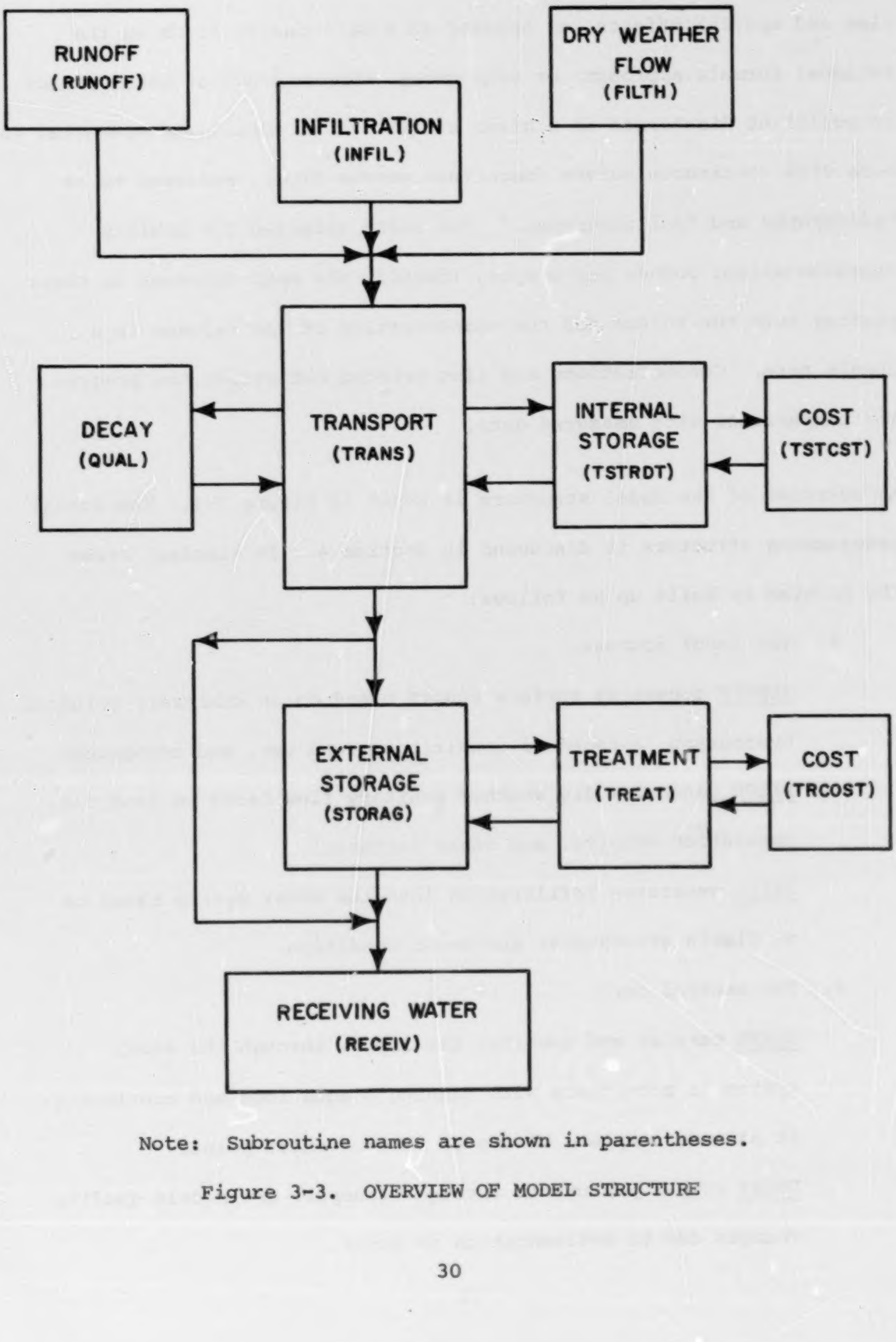
   FILTH generates dry weather sanitary flow based on land use, population density, and other factors.

   INFIL generates infiltration into the sewer system based on available groundwater and sewer condition.

2. The central core:

   TRANS carries and combines the inputs through the sewer system in accordance with Manning's equations and continuity; it assumes complete mixing at various inlet points.

   DECAY routes pollutants through transport and models quality changes due to sedimentation or scour.


RUNOFF
(RUNOFF)
INFILTRATION
(INFIL)
DRY WEATHER FLOW
(FILTH)
DECAY
(QUAL)
TRANSPORT
(TRANS)
INTERNAL STORAGE
(TSTRDT)
COST
(TSTCST)
EXTERNAL STORAGE
(STORAG)
TREATMENT
(TREAT)
COST
(TRCOST)
RECEIVING WATER
(RECEIV)

Note: Subroutine names are shown in parentheses.

Figure 3-3. OVERVIEW OF MODEL STRUCTURE

3. The correctional devices:

   TSTRDT, TSTCST, STORAG, TREAT, and TRCOST modify hydrographs and pollutographs at selected points in the sewer system, accounting for retention time, treatment efficiency, and other parameters; associated costs are computed also.

4. The receiving waters:

   RECEIV routes hydrographs and pollutographs through the receiving waters, which may consist of a stream, stream bed, lake, or estuary.

The quality constituents considered for simulation are the 5-day BOD, total suspended solids, total coliforms (represented as a conservative pollutant), and DO. These constituents were selected on the basis of available supporting data and importance in treatment effectiveness evaluation. Notable omissions, such as floatables, nutrients, and temperature, fell outside the scope of this initial work. Other parameters such as COD, volatile suspended solids, settleable solids, and fecal coliforms can be developed by paralleling the structures of their modeled counterparts.

## MODEL APPLICATIONS

The program is intended for use by municipalities, government agencies, and consultants as a tool for evaluating the pollution potential of existing systems, present and future, and for comparing alternate courses of remedial action. Although cost-effectiveness techniques are fully utilized in the Model analysis, the preponderance of human decisions inherent in this field of work precludes the achievement of an optimal solution. For example, the removal of one pollution unit from a receiving water will have different values to different people at different times.

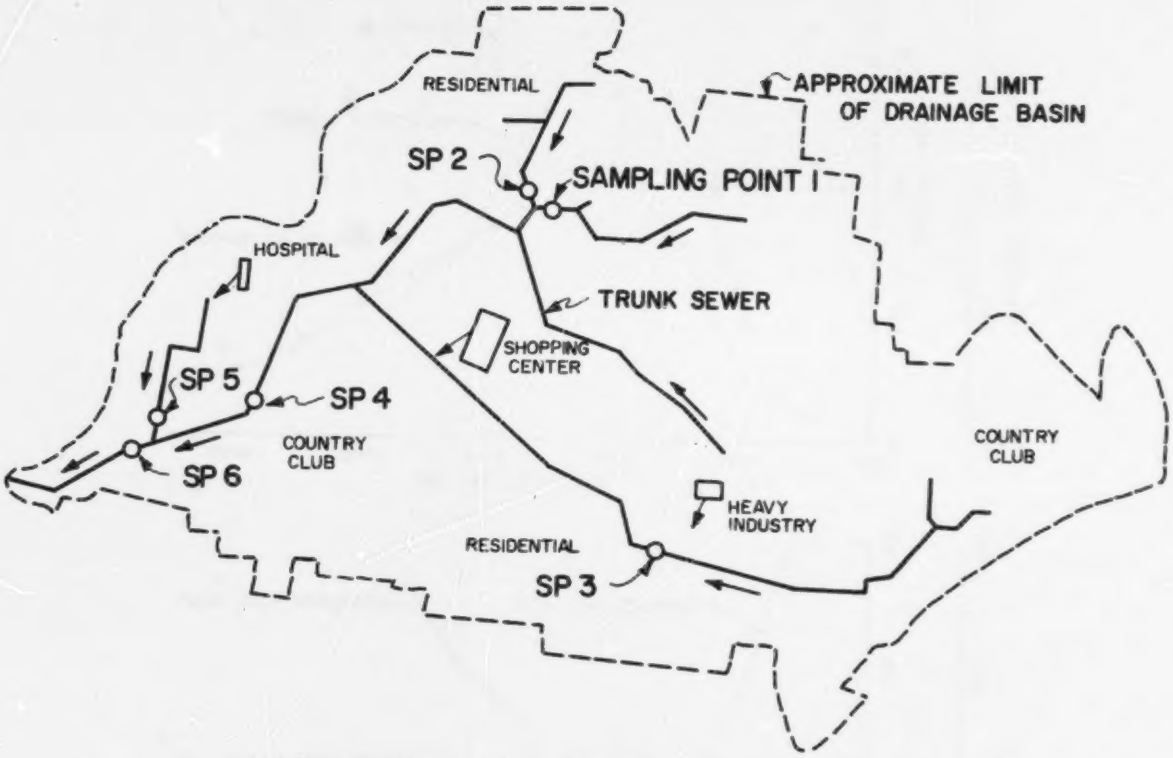
Further, the present limitations in the state of the art in defining the performance and costs of real corrective facilities must be realized when interpreting the results. The criteria used in the Model are based largely on concurrent EPA-sponsored Demonstration Grant activities and comparisons with conventional sewage (DWF) treatment facilities. Because of these restrictions the development emphasis has been placed on the basic structuring of the system phenomena with general credibility, rather than on the exact reproduction of numerical values in the few instances where such data are available.

Demonstration test data comparing modeled with measured results for a 2,600-acre combined sewered area in Cincinnati are shown in Figures 3-4 and 3-5 for DWF and combined sewage overflow respectively. Complete demonstration test data are presented in Volume II.

Programming considerations are discussed in Section 4.

## Users

A knowledge of FORTRAN programming and operating systems interfacing is essential for the initial setup for comprehensive runs on any new test area. Subsequent runs involving only input data changes, such as the input rainfall hyetographs or area characteristics, may be readily accomplished by engineers having only a brief familiarization with


RESIDENTIAL
APPROXIMATE LIMIT OF DRAINAGE BASIN
SP 2
SAMPLING POINT 1
HOSPITAL
TRUNK SEWER
SHOPPING CENTER
SP 5
SP 4
COUNTRY CLUB
SP 6
COUNTRY CLUB
HEAVY INDUSTRY
RESIDENTIAL
SP 3

SYSTEM MAP WITH SAMPLING LOCATIONS

DRY WEATHER FLOW RESULTS

| Sampling Location | Flow, cfs | | BOD, mg/L | | SS, mg/L | | Coli, MPN/100 ml |
|---|---|---|---|---|---|---|---|
| | Reported* | Computed | Reported | Computed | Reported | Computed | Computed |
| 1 | 0.93 | 0.90 | 360. | 403. | 224. | 206. | $9.5 \times 10^7$ |
| 2 | 0.54 | 0.50 | 350. | | 230. | | |
| 3 | 1.45 | 2.12 | 1,160. | | 236. | | |
| 4 | 15.50 | 12.58 | 618. | 529. | 265. | 226. | $7.0 \times 10^7$ |
| 5 | 0.50 | 0.80 | 292. | | 181. | | |
| 6 | 13.94 | 13.61 | 412. | 517. | 252. | 224. | $7.6 \times 10^7$ |

*Reported values are averages of approximately 10 grab samples each over a two week period.

Figure 3-4. BLOODY RUN DRAINAGE BASIN, CINCINNATI
DRY WEATHER FLOW RESULTS

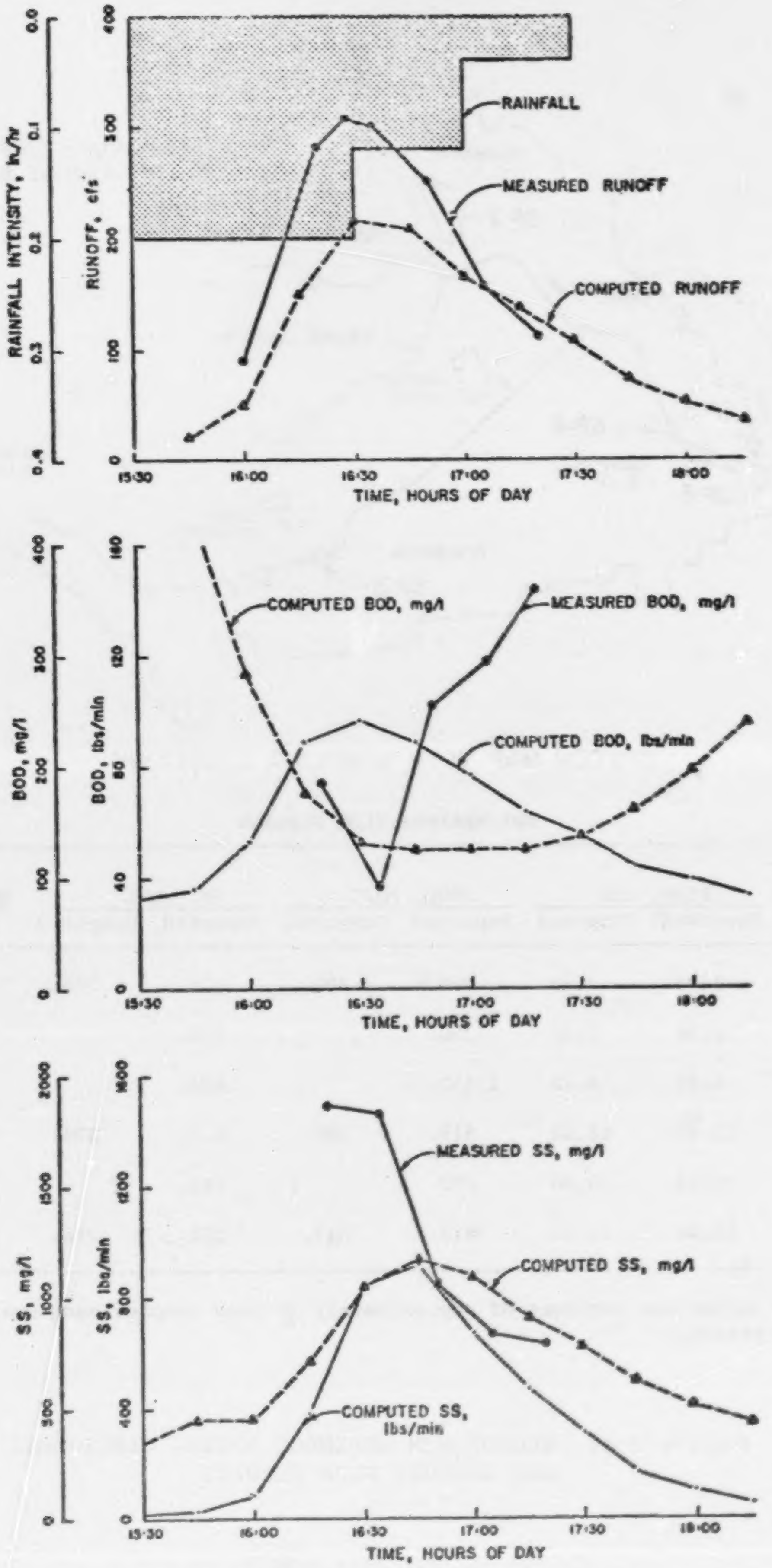


Figure 3-5. CINCINNATI COMBINED SEWER OVERFLOW RESULTS - STORM OF APRIL 1, 1970, SAMPLING POINT 3

computer programming, by following the step-by-step instructions furnished in the "User's Manual."

The ideal user team would consist of representatives of the city, for their knowledge of existing facilities and problems and probably responsibility for implementation; the consultant, for programming and development expertise; and the regulatory agency, for setting standards and basin-wide considerations.

## Data Requirements

A brief summary of generalized data requirements to support an application of the Storm Water Management Model is shown in Table 3-1. The measured hydrographs and characteristics referred to in Items 6 and 7 are used only for correlation-verification analyses. These measurements may be discontinued or omitted where satisfactory correlation has been established.

The data have been set up so that after the initial run only the storm (rainfall hyetographs), conditions of occurrence (clock time and antecedent history), and conditions of corrective action (storage/treatment) need to be revised.

For a 4,200-acre combined sewer area in the District of Columbia the total data requirement was described on approximately 500-600 punched cards, and only 10-20 percent of these required modification to handle multiple storm events and alterations in the system. By comparison there are over 10,000 card operations in the complete Model routine.

Table 3-1. GENERAL DATA REQUIREMENTS, STORM WATER MANAGEMENT MODEL

Item 1. Define the Study Area

Land use, topography, population distribution, census tract data, aerial photos, area boundaries.

Item 2. Define the System

Furnish plans of the collection system to define branching, sizes, and slopes. Types and general locations of inlet structures.

Item 3. Define System Specialties

Flow diversions, regulators, storage basins.

Item 4. Define System Maintenance

Street sweeping (description and frequency). Catchbasin cleaning. Trouble spots (flooding).

Item 5. Define the Receiving Waters

General description (estuary, river, or lake). Measured data (flow, tides, topography, water quality).

Item 6. Define the Base Flow (DWF)

Measured directly or through sewerage facility operating data. Hourly variation and weekday vs. weekend. DWF characteristics (composited BOD and SS results). Industrial flows (locations, average quantities, quality).

Item 7. Define the Storm Flow

Daily rainfall totals over an extended period (6 months or longer) encompassing the study events. Continuous rainfall hyetographs, continuous runoff hydrographs, and combined flow quality measurements (BOD and SS) for the study events. Discrete or composited samples as available (describe fully when and how taken).

## Programming Costs

Based on experience gained through the demonstration test series, it is anticipated that costs associated with the use of the Model for comprehensive studies will vary from ten thousand to several hundred thousand dollars per city, depending on the size, complexity, and depth of investigations. These costs may appear large unless compared with the millions and perhaps billions of dollars required to be spent to achieve proposed levels of water quality improvement.

# SECTION 4

## PROGRAMMING CONSIDERATIONS

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# SECTION 4

# PROGRAMMING CONSIDERATIONS

## PROGRAMMING LANGUAGE

This version of the EPA Storm Water Model consists of over 10,000 FORTRAN statements in the form of a main program and a large number of subprograms. FORTRAN was selected as the programming language because it is possible to run the program on machines from various manufacturers without changing the program. In order to meet this objective, certain restrictions were placed on the programming to ensure that machine-dependent features did not appear. Thus, the contractors agreed to use only ASA FORTRAN unless there was assurance that an extension of the language was available on a reasonable number of machines. As an example of such an extension, it was agreed that in format statements, single quote marks could be used to delimit Hollerith information, in addition to the standard nH format.

It was further agreed that the programming should be carried through in such a way that it would be comprehensible to each of the contractors and to qualified personnel from EPA or other interested agencies. Primarily this objective was met by the thorough documentation submitted in Volumes III and IV of this series. Additionally, certain programming practices were followed that were believed to be generally helpful, although not actually required by the rules of FORTRAN IV. Some of these were as follows:

1. Variable names should in general have mnemonic value.

2. Variable names used in various subroutines should be uniform for the most part throughout the Model.
3. Typing of FORTRAN variables should follow the standard default rule that those beginning with I, J, K, L, M and N are of type INTEGER.
4. High execution speed is desirable, but generally should be secondary to ease of understanding.
5. Numerical data and results in the printout should be sufficiently labeled so that reference to the FORTRAN program for identification is unnecessary.
6. Extensive commenting should be provided throughout the FORTRAN program.

## PROGRAM BLOCKS

At the outset of the project it was expected that, machine storage permitting, the whole program would be run at once, corresponding to every physical event from a rainstorm falling on a watershed down to computing details of events in the receiving water. If machine storage proved insufficient, the expectation was that overlay techniques could be used. As the programming effort went forward, however, it became clear that there were distinct advantages in keeping the major parts of the program as separate entities, which are hereafter referred to as "blocks."

The adopted arrangement, as shown in Figure 4-1, consisted of a main control and service block, the Executive Block, and four computational blocks: (1) Runoff Block, (2) Transport Block, (3) Storage Block, and (4) Receiving Water Block.

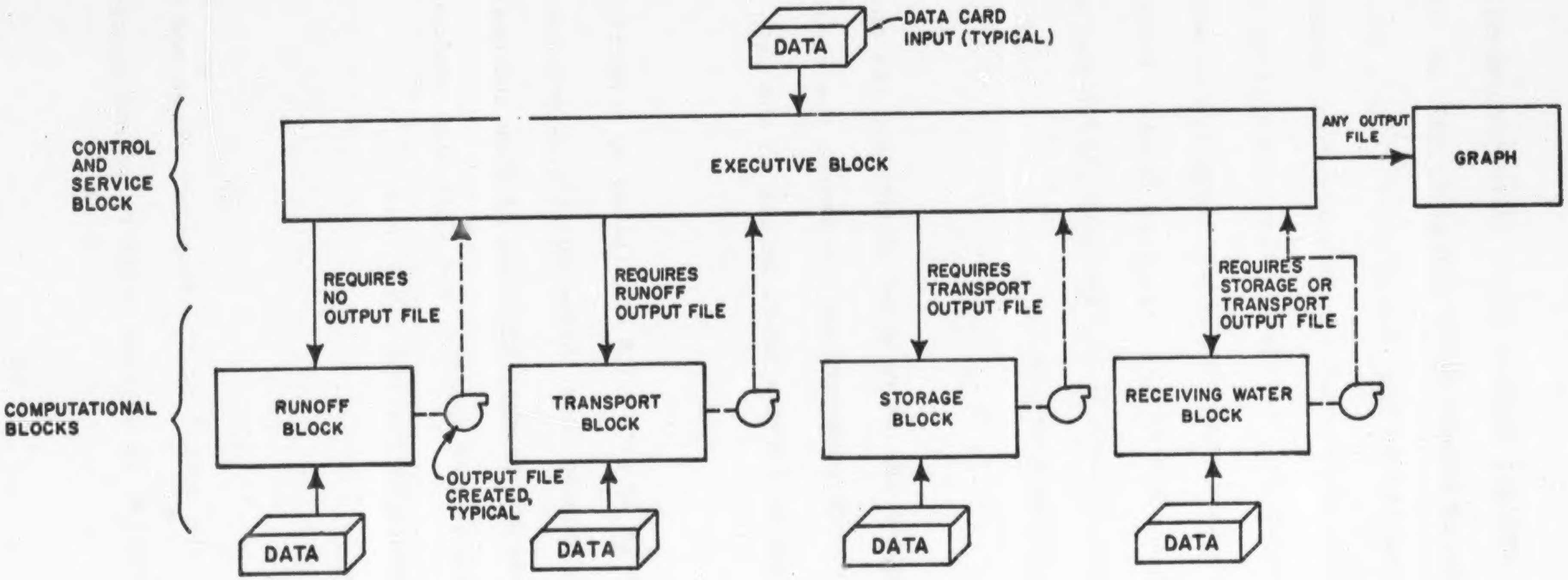
DATA
DATA CARD INPUT (TYPICAL)
CONTROL AND SERVICE BLOCK
EXECUTIVE BLOCK
ANY OUTPUT FILE
GRAPH
REQUIRES NO OUTPUT FILE
REQUIRES RUNOFF OUTPUT FILE
REQUIRES TRANSPORT OUTPUT FILE
REQUIRES STORAGE OR TRANSPORT OUTPUT FILE
COMPUTATIONAL BLOCKS
RUNOFF BLOCK
TRANSPORT BLOCK
STORAGE BLOCK
RECEIVING WATER BLOCK
OUTPUT FILE CREATED, TYPICAL
DATA
DATA
DATA
DATA

Figure 4-1. MASTER PROGRAMMING ROUTINE

## Executive Block

The Executive Block assigns logical units (disk/tape/drum), determines the block or sequence of blocks to be executed, and, on call, produces graphs of selected results on the line printer. Thus, this Block does no computation as such, while each of the other four blocks are set up to carry through a major step in the quantity and quality computations. All access to the computational blocks and transfers between them must pass through subroutine MAIN of the Executive Block. Transfers are accomplished on off-line devices (disk/tape/drum) which may be saved for multiple trials or permanent record.

## Runoff Block

The Runoff Block computes the storm water runoff and its characteristics for a given storm for each subcatchment and stores the results in the form of hydrographs and pollutographs at inlets to the main sewer system.

## Transport Block

The Transport Block sets up pre-storm conditions by computing DWF and infiltration and distributing them throughout the conveyance system. The Block then performs its primary function of flow and quality routing, picking up the runoff results, and producing combined flow hydrographs and pollutographs for the total drainage basin and at selected intermediate points.

## Storage Block

The Storage Block uses the output of the Transport Block and modifies the flow and characteristics at a given point or points according to the predefined storage and treatment facilities provided. Costs associated with the construction and operation of the storage/treatment facilities are computed.

### Receiving Water Block

The Receiving Water Block accepts the output of the Transport Block directly, or the modified output of the Storage Block, and computes the dispersion and effects of the discharge in the receiving river, lake, or bay.

In principle, the capability exists to run all blocks together in a given computer execution, although from a practical and sometimes necessary (due to computer core limitations) viewpoint, typical runs involve one or two computational blocks together with the Executive Block. Using this approach avoids overlay and, moreover, allows for examination of intermediate results before continuing the computations. Further, it permits the use of intermediate results as start-up data in subsequent execution runs, thereby avoiding the waste of repeating the computations already performed.

## MACHINE REQUIREMENTS AND COMPATIBILITY

All parts of the program have been run on at least two machines. During development of the program M&E used an IBM 360/67 at the Stanford University Computation Center; WRE used a UNIVAC 1108 located in Oakland and accessed by their terminal in Walnut Creek; and UF initially used an IBM 360/50 until it was replaced at the University of Florida Computation Center by an IBM 360/65.

In addition to using these various machines with their standard compilers, M&E tested all parts of the program using the FORTRAN compiler developed by the University of Waterloo (WATFOR). This compiler uncovered a number of "bugs" that the IBM FORTRAN IV compilers were willing to tolerate. Corrections were made accordingly to avoid possible difficulties if the program were to run in the future on a different type of machine. The most common bug uncovered by WATFOR was failure to set initial values for all variables.

## STORAGE AND DATA FILES

As it stands, the program can be run on a machine having core storage capacity of at least 350K bytes (or equivalent). In addition, the program uses peripheral storage devices which may consist of disk, tape, or drum units, depending on the machine configuration. The UNIVAC 1108 used by WRE typically has tape and drum, while the IBM 360 systems used by M&E, UF, and EPA have disk capabilities that make it possible to avoid tape usage. One IBM 2314 Disk Storage Device can provide ample room for all files created by the program, or alternatively, the files can be placed on several devices, depending on the system configuration. The disk units are used both for transfer of information between blocks, in which case data files are created, and for scratch files used only temporarily in execution of a given block.

A sample of core capacities used by separate program blocks on representative demonstration runs is shown in Table 4-1.

Table 4-1. REPRESENTATIVE VALUES OF CORE CAPACITIES REQUIRED IN DEMONSTRATION RUNS

| Program Block(s) | Machine Core Storage Required, bytes |
|---|---|
| Executive* | 225K |
| Executive and Runoff* | 264K |
| Executive and Transport* | |
| Without internal storage | 310K |
| With internal storage | 334K |
| Storage and Treatment | 183K |
| Executive and Receiving Water* | 327K |

*Of the storage required, 170K bytes are required in the common blocks.

The data files correspond to a given storm on a given site, and in many cases they can be used repeatedly to test alternatives without going through the entire computation each time. Although the original intention was to use formatting on these files, it has proven preferable to use unformatted input/output which greatly speeds up read/write operations.

Program Input

In addition to the files described above, the various blocks of the program use data card input as described in the "User's Manual," Volume III of this series. Thus, the user must prepare the data cards needed for a given block and also supply JCL cards corresponding to the appropriate file devices.

It is also possible with some systems to hold the data card images on a file. If, in addition, the FORTRAN (or its object module) is held on a file, the program can be run with no card input. This method has been used in conjunction with terminal access on the IBM 360/67 at Stanford University. With so large a program the elimination of card reading is very convenient.

## Program Compile and Execution

A sample of the compile and execution times used by separate program blocks on representative demonstration runs, IBM 360/67, is shown in Table 4-2. This table further illustrates the savings which were made by storing compiled blocks of the program in a permanent job library (Load Modules).

No general attempt was made for overall optimization of the program with regard to storage or timing. It is very likely that significant improvements can be made, although the present requirements are not considered excessive in view of the amount of information generated. Some initial versions of the Model were restructured to improve timing by eliminating the need for SORT and MERGE operations on peripheral devices. Also, the input and output operations onto data files are unformatted to speed up the read/write operations. Again, it was agreed to avoid the adjustable dimensioning feature of FORTRAN IV since it is known to be time-consuming.

Table 4-2. SAMPLE COMPILE AND EXECUTION TIMES ON DEMONSTRATION RUNS

| | Machine time, min | | | |
|---|---|---|---|---|
| | Uncompiled* | | Load Module** | |
| Program Block(s) | CPU*** | Execution Time**** | CPU*** | Execution Time**** |
| Executive | -- | -- | 0.18 | 0.28 |
| Executive and Runoff | 1.39 | 2.22 | 1.15 | 1.97 |
| Executive and Transport (without internal storage) | 2.77 | 4.38 | 0.72 | 1.16 |
| Executive and Transport (with internal storage) | 3.06 | 4.71 | 0.85 | 1.30 |
| Storage and Treatment | 0.36 | 0.52 | -- | -- |
| Receiving Water | 2.60 | 3.57 | -- | -- |

*Time includes compile, link-edit, and execute.

**Time required for link-edit and execute only.

***Actual computational time in computer core not including the time needed to execute the read and write (I/O) statements or to run the peripheral devices.

****Time required in the computer including I/O statements.

## Program Output

Output from the program consists of the usual line printer tabulations, which, if desired, can be supplemented by selected plotted hydrographs and pollutographs also produced on the line printer at execution time. These plots are available through subroutine GRAPH of the Executive Block, using the output files of any of the computational blocks.

There has been discussion from time to time of incorporating improved offline plotter capability in the program, but it was unfeasible to do so within the scope of the current project. In any event, such plotting would use output files of the kind now generated, and could be added as a set of subroutines to be called by the Executive Block.

# PART 2

# QUANTITY (HYDROLOGIC) SUBROUTINES

# SECTION 5

## SURFACE RUNOFF QUANTITY MODEL

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# SECTION 5

## SURFACE RUNOFF QUANTITY MODEL

### OBJECTIVES

The Surface Runoff Quantity model (subroutine RUNOFF) simulates the runoff phenomena of a drainage basin for any given rainfall pattern. The objective of the model is to provide a basis for subsequent computation of pollutographs, a time-history of runoff quality, that are necessary for planning, design, and operation of the Storm Water Management Model.

The EPA has contemplated the use of the Model throughout the United States. Probably the only information that can reasonably be expected to be available in any locality is climatological data and the physical watershed characteristics, such as size, ground slope, and types of ground cover.

To meet this specific objective, RUNOFF represents the drainage basin by an aggregate of idealized subcatchments and gutters that are common denominators of the drainage plan and are readily quantifiable. Flexibility of the program to be adopted to any prototype conditions was of great concern and was considered in the model formulation and development.

The model accepts the input of any rainfall hyetograph, a time-history of rainfall intensity, as given. The rainfall hyetograph for a location is more readily available from the Weather Bureau and is subjected to less influence by engineering works. Its prediction by stochastic process is not part of the Surface Runoff Quantity model.

## RELATION TO STORM WATER PROGRAM

RUNOFF is the first computational subroutine of the Storm Water Management Model. It calculates the hydrographs by a step-by-step accounting of rainfall, infiltration, detention, overland flow, and gutter flow. In so doing, it provides information about the overland flow and gutter flow at various parts of the drainage system for use by the Surface Runoff Quality model. It also generates hydrographs for the designated entry points to the major sewer line. These hydrographs are then accepted by the Transport Model which routes the quantity and quality of wastewater throughout the sewerage system, to the treatment plant, and to the receiving water body.

The most important function of RUNOFF is therefore to furnish hydraulic information at the right time and right place to serve the need of other models. While the Model can get started by itself, its answers must be quite accurate and compatible with those of other models.

## THE MODEL SUBROUTINES

The computer program developed to perform the Surface Runoff Quantity model computation consists of six subroutines, i.e., RUNOFF, HYDRO, RHYDRO, WSHED, GUTTER, and HCURVE. Detailed documentations of each subroutine are provided in the "User's Manual." Only a brief description of their functional relationships is discussed herein.

Subroutine RUNOFF provides the necessary liaison with the master program of the overall Storm Water Management Model. RUNOFF, in turn, calls

HYDRO to coordinate the computation of the hydrograph with the assistance of three core subroutines, RHYDRO, WSHED, and GUTTER.

Subroutine RHYDRO is called by HYDRO to read all input data related to the drainage basin and to perform initial preparatory work such as unit conversion and error detection. Default options are available for the typical values of physical coefficients describing the roughness factor, detention requirement, and infiltration characteristics. A normal execution of RHYDRO provides all the necessary information for the calculation of a runoff hydrograph.

Subroutine WSHED computes the depth and flow rate of water overland. It interpolates the rainfall hyetograph to obtain the rainfall intensity for the period of computation and accounts for infiltration and surface storage. Upon completion, it returns with a set of water depth and flow for the subcatchments.

The function of subroutine GUTTER is similar to that of WSHED. It calculates a complete set of water depth and flow for the gutters and pipes in the system. It accounts for surcharge in full conduits, assuming temporary storage at the upstream junction until surcharge is no longer necessary.

The hydrograph solutions are printed in tabulated form. In addition, HCURVE will prepare X-Y coordinates of the hydrograph at selected points of the drainage system for printer plot routines which plot the curves directly in the computer output.

There is a call to subroutine GRAPH (0) from subroutine RUNOFF. GRAPH is described in Section 4. The Surface Runoff Quality model, subroutine SFQUAL in RUNOFF, is described in Section 11.

## THEORETICAL DEVELOPMENT

Existing methods of runoff estimate were evaluated as background and for potential use in the contemplated model. Methods evaluated include the rational method; "rational-rational" method (Ref. 1); British Road Research Laboratory Method (Ref. 2); dimensionless hydrograph method of Izzard (Refs. 3,4); Chicago Method (Ref. 5); and the Stanford Watershed Model (Ref. 6).

Empiricism at various degrees was found in all the methods that used factors not easily determined by any ready rule. However, previous studies have advanced much of the theoretical ground work related to the fundamental process of runoff phenomena, i.e., rainfall, infiltration, detention, overland flow, and gutter flow. For the development of a comprehensive surface runoff quantity model, each of these processes must be simulated individually and operated simultaneously by a computer program.

### Model Geometry

Model geometry is a discretization procedure for the mathematical abstraction of the physical drainage system. For the computation of the hydrograph, the drainage basin may be conceptually represented by a network of hydraulic elements, i.e., subcatchments, gutters, and pipes. Hydraulic properties of each element are then characterized by various parameters, such as size, slope, and roughness coefficient.

As was originally envisioned, a subcatchment is rectangular in shape with reasonably uniform watershed characteristics, such as surface cover and ground slope. The geometry of a subcatchment is defined by the area, width, and ground slope. The type of ground cover determines the detention depth requirements, the roughness factor (such as Manning's coefficient), and the coefficients describing the infiltration loss by Horton's exponential function (Ref. 5).

The subcatchments need not be the same size and the irregular shape can be approximated by an equivalent rectangle by computing the mean width. In principle, a series of subcatchments can be designed to cover the entire drainage basin.

The subcatchments form an aggregated system by gutters and pipes specifying the connectivity of flow. Hydraulically, gutters and pipes are described by the Manning's coefficient, length, invert slope, and geometric description of the shape. The latter may include the bottom width and side slopes for rectangles, trapezoids, and triangles and the diameter for circular pipes.

Figure 5-1 presents a typical but idealized drainage system. As shown, subcatchment 2 drains into gutter 1. Both gutter 1 and subcatchment 1 discharge into gutter 2. Several subcatchments and gutters can be connected to a given gutter, depending on the topographic condition.

## Solution Procedure

Once the properties of each hydraulic element are given, the computer can be instructed to make a step-by-step accounting of how much water

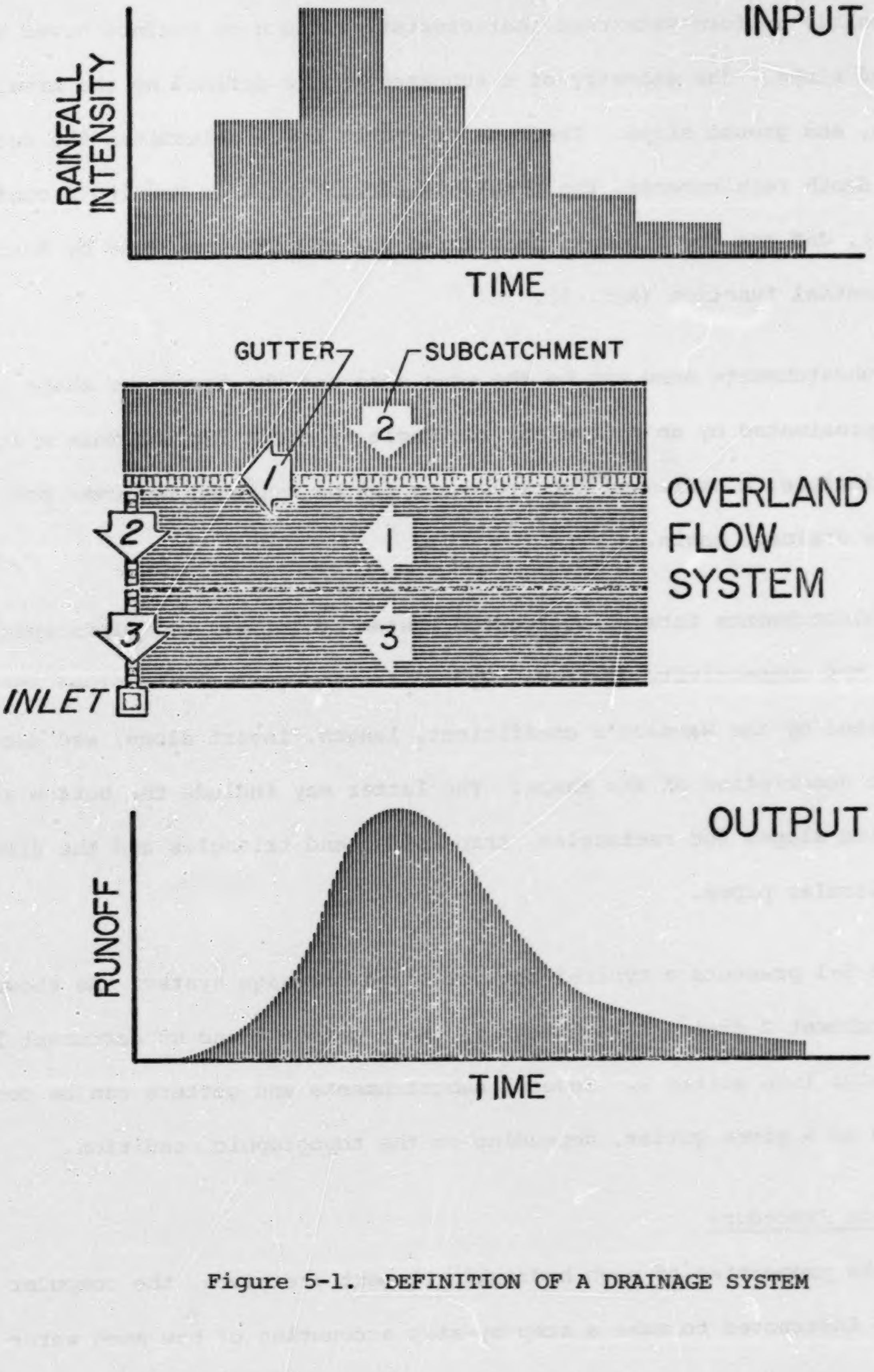

INPUT
RAINFALL INTENSITY
TIME
GUTTER
SUBCATCHMENT
2
1
2
1
3
3
OVERLAND FLOW SYSTEM
INLET
OUTPUT
RUNOFF
TIME

Figure 5-1. DEFINITION OF A DRAINAGE SYSTEM

comes in, how much is lost to infiltration, and what will be the outflow and the remaining water depth in each increment of time. This accounting procedure can best be described by the flow chart shown in Figure 5-2. First, the computer reads and edits data pertaining to the rainfall hyetograph, subcatchment, and gutter characteristics and their interconnections. Unit conversion is made for all parameters to conform in the pound-foot-second system.

Stepwise computation proceeds as follows (Figure 5-2):

1. Rainfall is added to the subcatchment according to the specified hyetograph,

$$D_1 = D_t + R_t \Delta t \tag{1}$$

where $D_1$ = Water depth after rainfall

$D_t$ = Water depth of the subcatchment at time, t

$R_t$ = Intensity of rainfall in time interval, $\Delta t$

2. Infiltration is computed by Horton's exponential function and is subtracted from water depth existing on the subcatchment,

$$I_t = f_o + (f_i - f_o)\, e^{-\alpha t} \tag{2}$$

and

$$D_2 = D_1 - I_t \Delta t \tag{3}$$

where $f_o$, $f_i$, and $\alpha$ are coefficients in Horton's equation $D_2$ is intermediate water depth after accounting for infiltration.

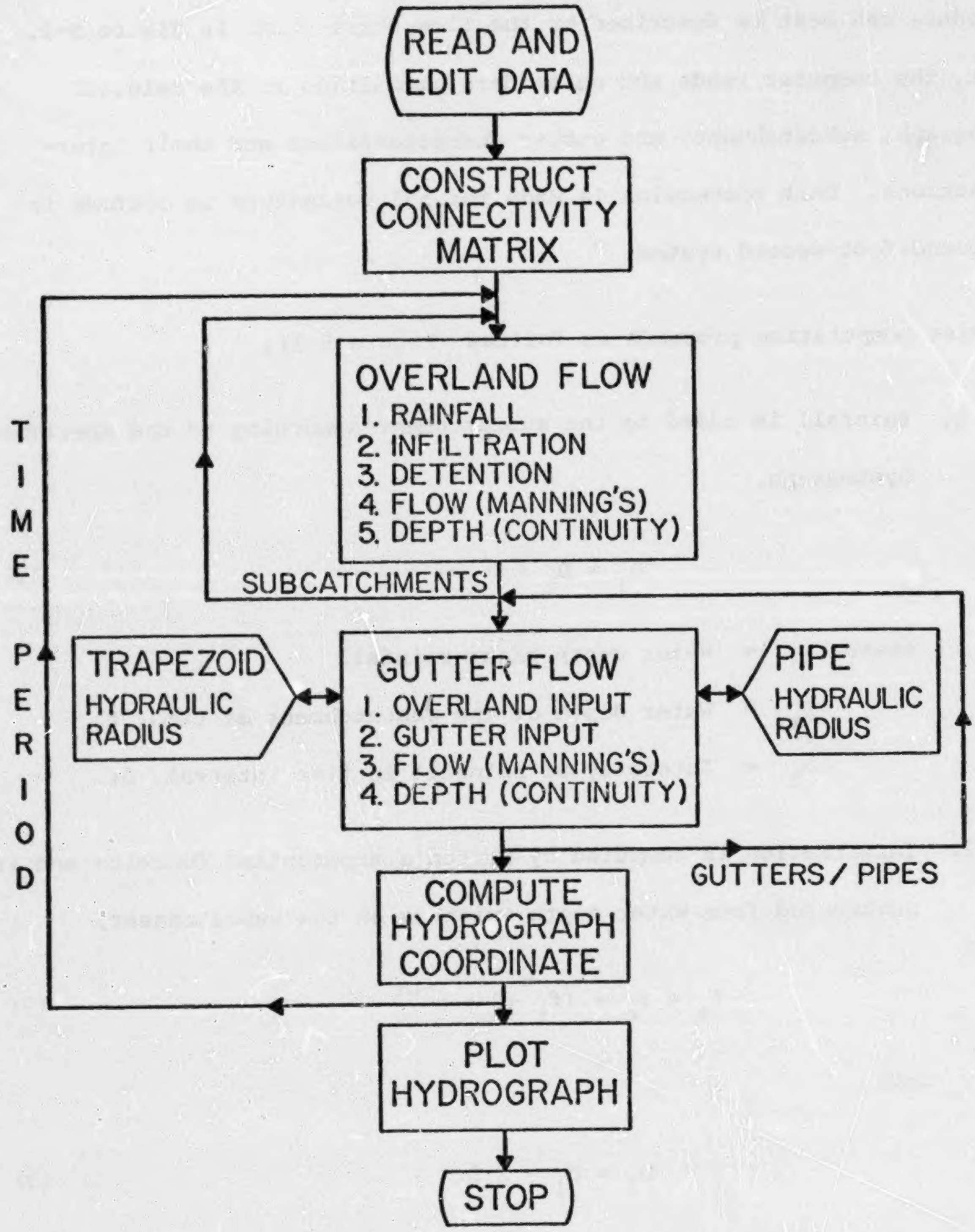

READ AND EDIT DATA
CONSTRUCT CONNECTIVITY MATRIX
OVERLAND FLOW
1. RAINFALL
2. INFILTRATION
3. DETENTION
4. FLOW (MANNING'S)
5. DEPTH (CONTINUITY)
SUBCATCHMENTS
TIME PERIOD
TRAPEZOID
HYDRAULIC RADIUS
GUTTER FLOW
1. OVERLAND INPUT
2. GUTTER INPUT
3. FLOW (MANNING'S)
4. DEPTH (CONTINUITY)
PIPE
HYDRAULIC RADIUS
GUTTERS / PIPES
COMPUTE HYDROGRAPH COORDINATE
PLOT HYDROGRAPH
STOP

Figure 5-2. FLOW CHART, HYDROGRAPHIC COMPUTATION

3. If the resulting water depth of the subcatchment, $D_2$, is larger than the specified detention requirement, $D_d$, an outflow rate is computed using Manning's equation,

$$V = \frac{1.49}{n} (D_2 - D_d)^{2/3} S^{1/2} \tag{4}$$

and

$$Q_w = V W (D_2 - D_d) \tag{5}$$

where $V$ = Velocity

$n$ = Manning's coefficient

$s$ = Ground slope

$W$ = Width

$Q_w$ = Outflow rate

4. The continuity equation is solved to determine the water depth of the subcatchments, resulting from the rainfall, infiltration, and outflow,

$$D_{t+\Delta t} = D_2 - (Q_w/A)\Delta t \tag{6}$$

where $A$ is the surface area of the subcatchment.

5. Steps 1 to 4 are repeated until computations for all subcatchments are completed.

6. The inflow ($Q_{in}$) to a gutter is computed as a summation of outflow from tributary subcatchments ($Q_{w,i}$) and flow rate of immediate upstream gutters ($Q_{g,i}$).

$$Q_{in} = \Sigma Q_{w,i} + \Sigma Q_{g,i} \tag{7}$$

7. The inflow is added to raise the existing water depth of the gutter according to its geometry,

$$Y_1 = Y_t + (Q_{in} / A_s) \Delta t \tag{8}$$

where $Y_1$ and $Y_t$ = Water depth of the gutter

$A_s$ = Mean water surface area between $Y_1$ and $Y_t$

8. The outflow is calculated for the gutter using Manning's equation,

$$V = \frac{1.49}{n} R^{2/3} S_i^{\;1/2} \tag{9}$$

and

$$Q_g = V A_c \tag{10}$$

where $R$ = Hydraulic radius

$S_i$ = Invert slope

$A_c$ = Cross-sectional area at $Y_1$

9. The continuity equation is solved to determine the water depth of the gutter, resulting from the inflow and outflow.

$$Y_{t+\Delta t} = Y_1 + (Q_{in} - Q_g) \Delta t / A_s \tag{11}$$

10. Steps 6 to 9 are repeated until all the gutters are finished.

11. The flows, reaching the point of concern, are added to produce a hydrograph coordinate along the time axis.

12. The processes from Steps 1 to 11 are repeated for succeeding time periods until the complete hydrograph is computed.

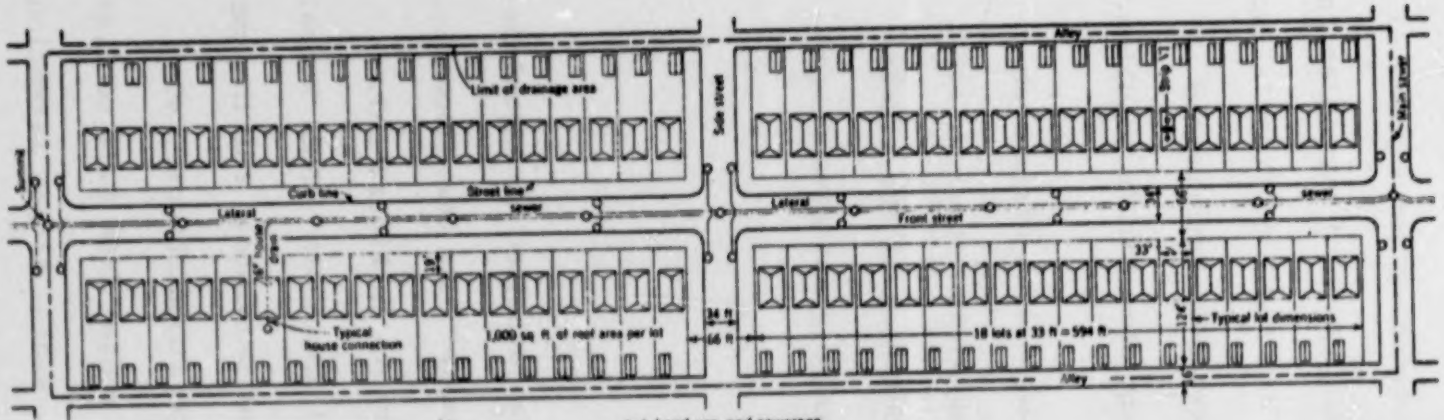
The step-by-step description of the solution procedure gives a physical picture of the processes being modeled. The integration of variables in each time increment was originally performed by the modified Euler's two-step method. This is now accomplished by the Newton-Raphson method (Ref. 7), which produces a smoother hydrograph and more stable solution.

As was noted, Manning's equation was used for computation during each time interval of integration. The "state" of the system, however, is being updated continuously. Thus, the dynamic behavior of the runoff phenomena was simulated by a stepwise and successive quasi-steady state approximation.

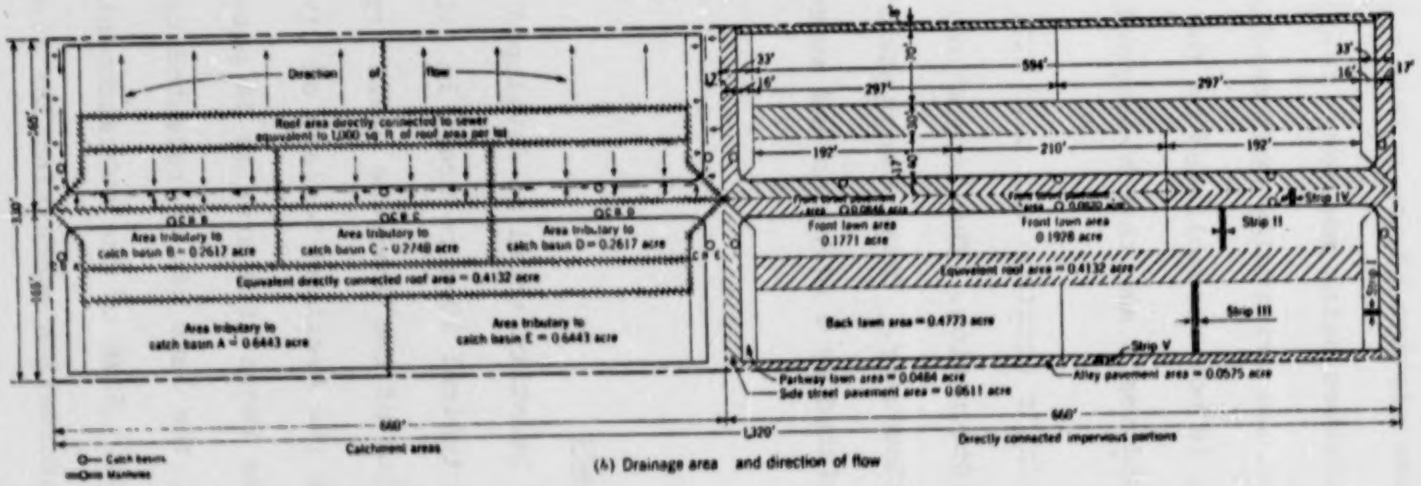
## TEST APPLICATIONS

### Chicago 10-Acre Tract

The Surface Runoff Quantity model as developed was utilized to calculate a hydrograph for a typical 10-acre tract in Chicago. Land use, geometry, and direction of flow of the drainage basin are shown in Figure 5-3, taken directly from the American Society of Civil Engineers (ASCE) Manual No. 37 (Ref. 8). As shown, the area is 54 percent pervious. For the purpose of simulation, the land was represented by 80 subcatchments, 40 gutters, and 4 pipes. Size of the subcatchments ranges from 0.04 to 0.48 acre. Necessary information concerning the size, width, slope, retention depth, and rate of infiltration was obtained from the manual.


Alley
Limit of drainage area
Side street
Main street
Summit
Strip VI
Curb line
Street line
Lateral
Sewer
Front street
House drain
16'
33'
34'
124'
10'
Typical house connection
1,000 sq. ft. of roof area per lot
34 ft
66 ft
18 lots at 33 ft = 594 ft
Typical lot dimensions

(a) Land use and sewerage


Direction of flow
Roof area directly connected to sewer
equivalent to 1,000 sq. ft. of roof area per lot
C.B. B
C.B. C
C.B. D
C.B. E
Area tributary to catch basin B = 0.2617 acre
Area tributary to catch basin C = 0.2748 acre
Area tributary to catch basin D = 0.2617 acre
Equivalent directly connected roof area = 0.4132 acre
Area tributary to catch basin A = 0.6443 acre
Area tributary to catch basin E = 0.6443 acre
330'
165'
1,320'
660'
Catchment areas
Catch basins
Manholes
33'
16'
17'
594'
297'
192'
210'
Front street pavement area = 0.0666 acre
Front street pavement area = 0.0630 acre
Front lawn area 0.1771 acre
Front lawn area 0.1978 acre
Strip II
Strip III
Strip IV
Strip V
Equivalent roof area = 0.4132 acre
Back lawn area = 0.4773 acre
Parkway lawn area = 0.0484 acre
Side street pavement area = 0.0611 acre
Alley pavement area = 0.0575 acre
Directly connected impervious portions

(b) Drainage area and direction of flow

Source: American Society of Civil Engineers, Manual of Engineering Practice No. 37, 1960 (Ref. 8).

Figure 5-3. TYPICAL CHICAGO 10-ACRE TRACT DRAINAGE BASIN

A 181-minute design storm was applied to the drainage basin. The rainfall hyetograph and the calculated runoff hydrograph were plotted in Figure 5-4. For the purpose of comparison, the calculated hydrograph by the Chicago Method is also shown in the figure. Results indicate that these hydrographs have a comparable rising limb. Both of them peak at about the same time, i.e., 75 minutes from the start of rainfall. However, the Chicago Method calculates a lower peak value (18 cfs against 22 cfs) and also a lower recession.

Since both curves are for the hypothetical case, the question of accuracy is not an issue. However, the Chicago Method is seen to produce a smaller total runoff. This can only be accounted for by a larger infiltration loss which might seem anomalous, because both methods use the same Horton's equation for calculating infiltration loss. The explanation is that the Chicago Method computed the mass curve for infiltration loss. Thus, the method tends to satisfy the infiltration rate at any given time even though there is not sufficient rainfall at that moment. The present method allows for the infiltration rate as calculated only when there is sufficient water on the ground at that given instant.

The infiltration loss of the present method was 28 percent of the total rainfall. The mass continuity has been maintained within 0.1 percent of the rainfall. No such information was available on the Chicago Method for comparison.

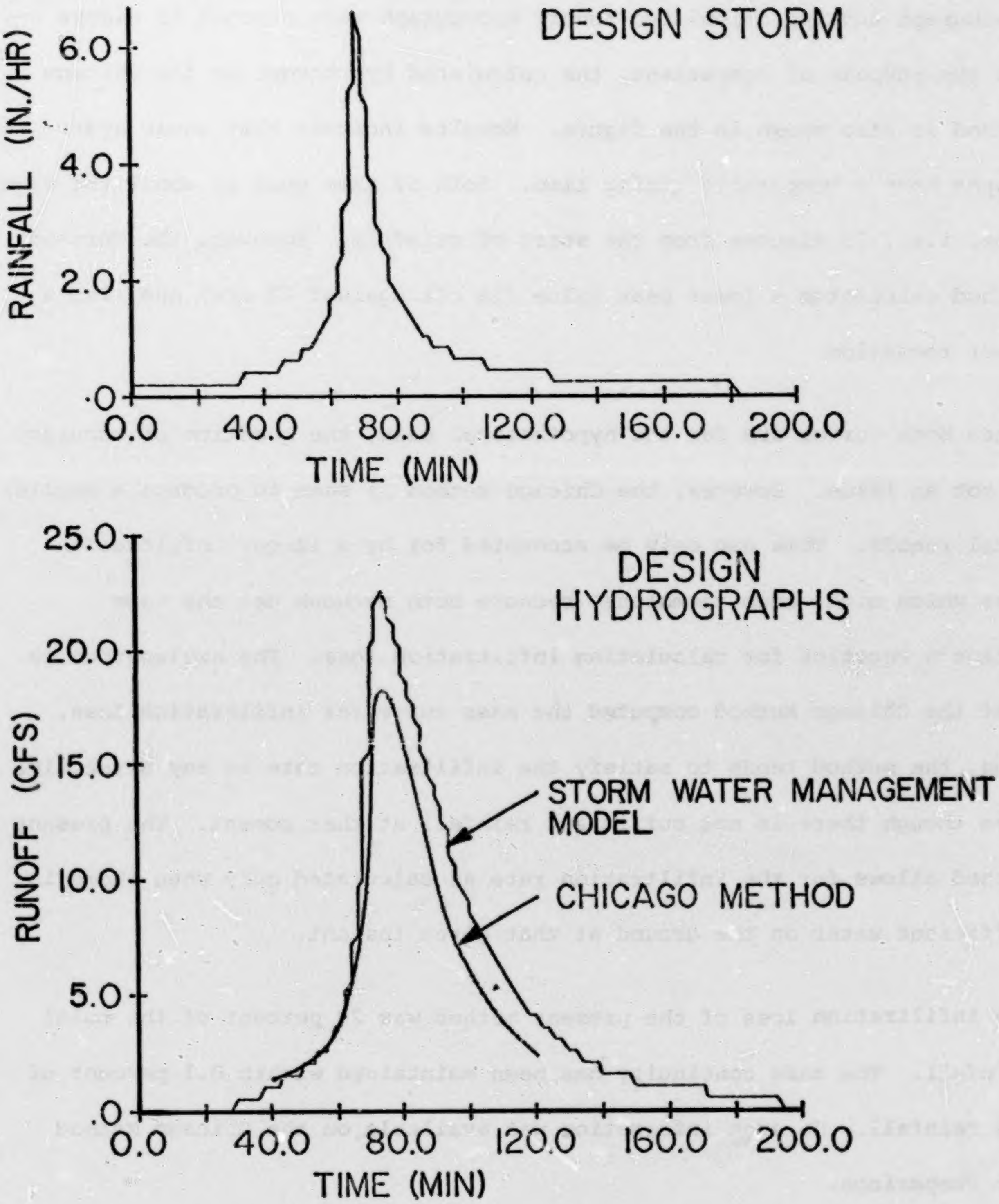


Figure 5-4. RAINFALL HYETOGRAPH AND CALCULATED RUNOFF HYDROGRAPHS, CHICAGO 10-ACRE TRACT

### Oakdale (Chicago)

Some rainfall and runoff data have been gathered at Oakdale (Chicago) for a drainage basin of 12.9 acres (Ref. 9). The land use is similar to the typical 10-acre tract described previously in Figure 5-3.

A detailed description of this particular drainage basin is not available. It was suggested that the Oakdale rainfall hyetograph could be applied on the typical 10-acre tract. The resulting runoff hydrograph could be multiplied by 1.29 to account for the difference in areas. This calculated runoff hydrograph could then be compared to the measured Oakdale hydrograph.

The calculated and measured runoff hydrographs are compared in Figure 5-5. The measured peak was found to be 2 to 3 cfs higher than the calculated one. The timing for the first and the second peaks was also noted to be offset by 5 and 10 minutes respectively. The overall comparison, however, is reasonably close.

### Northwood (Baltimore)

The Johns Hopkins University has conducted an intensive data gathering program for rainfall-runoff relationships at Northwood (Baltimore). The data for the verification of the model were taken from an ASCE Urban Water Resources Research Program publication (Ref. 10).

Briefly, the drainage basin area is 47.4 acres with approximately 60 percent as residential area and 40 percent as a shopping center, including a large parking lot. Average ground slope is 3 percent and the

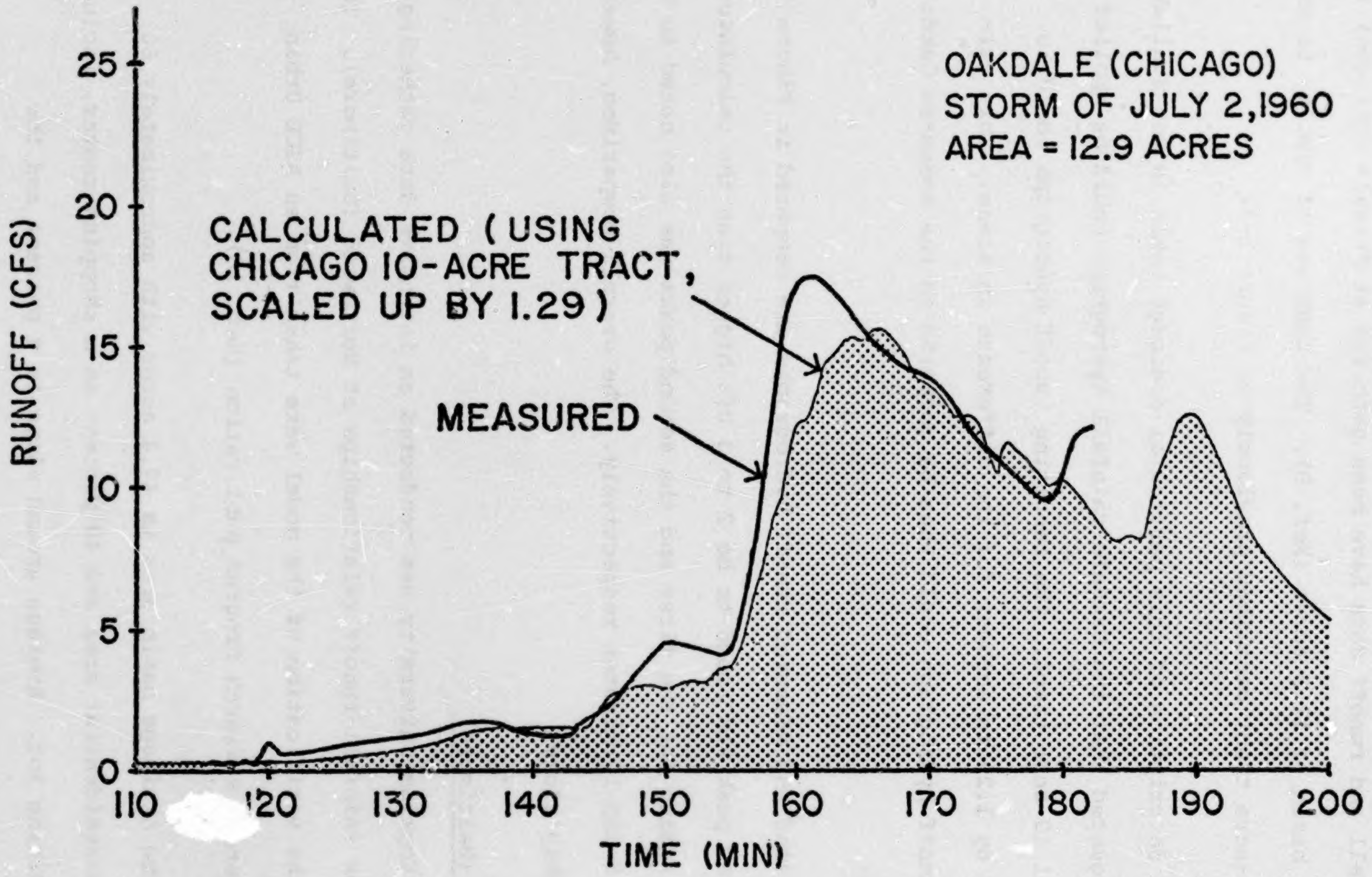


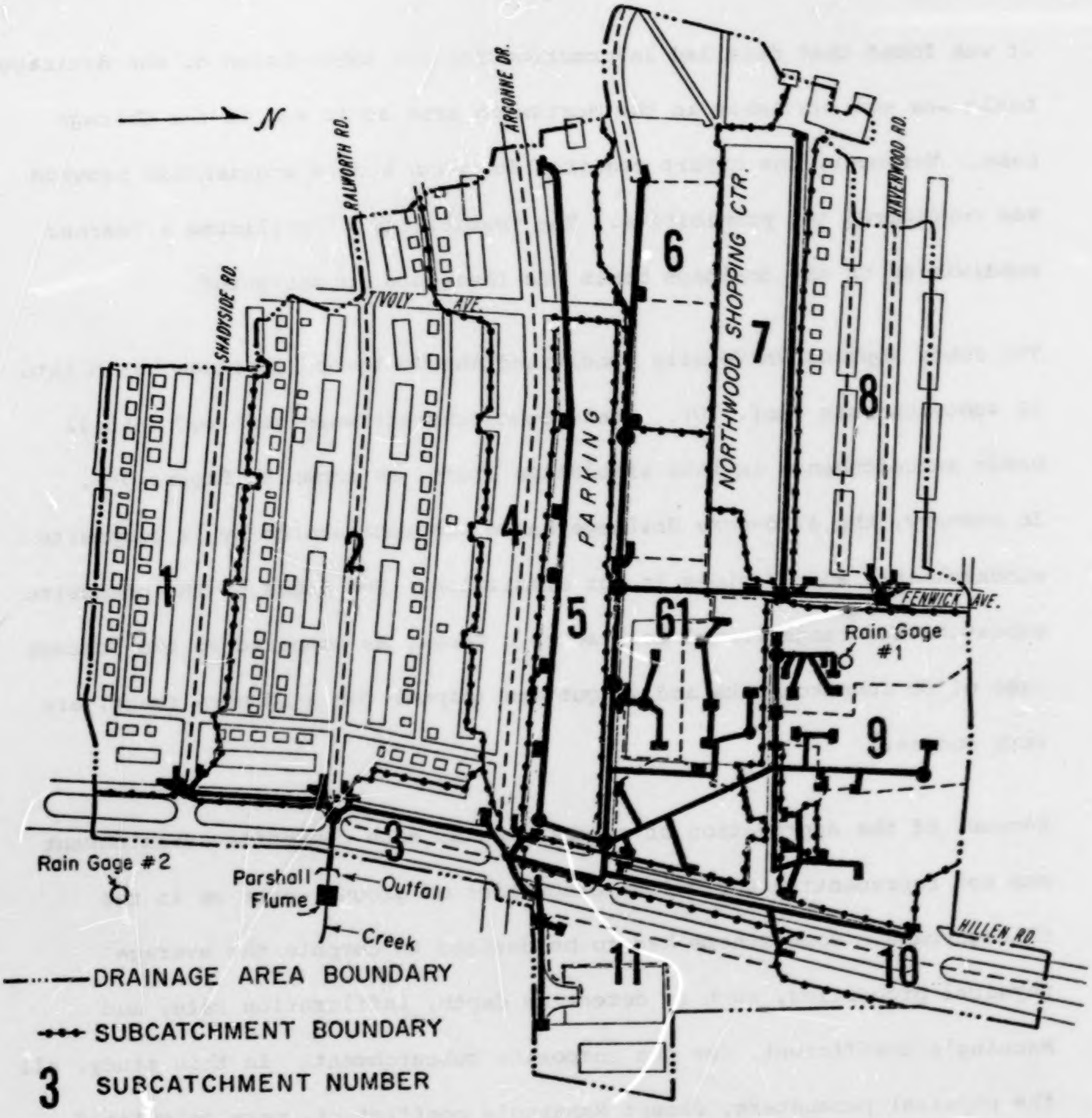
Figure 5-5. CALCULATED AND OBSERVED RUNOFF HYDROGRAPHS, OAKDALE (CHICAGO)

imperviousness is 68 percent. Other pertinent information concerning the drainage basin is given in the publication.

It was found that detailed information for the subdivision of the drainage basin was not available in the Northwood area as it was in the Chicago case. Moreover, the effort required for such a data acquisition program was considered too prohibitive. The feasibility of utilizing a coarser subdivision of the drainage basin was therefore investigated.

The Johns Hopkins University subdivided the Northwood drainage basin into 32 subcatchments (Ref. 10). These subcatchments were combined into 12 basic subcatchments in this simulation study, as shown in Figure 5-6. In summary, the 47.4-acre drainage basin was represented by 12 composite subcatchments and 13 pipes in the simulation. The areas of the composite subcatchments range from 2 to 9 acres. These, as compared to the Chicago case of 80 subcatchments and 44 gutters (pipes) for a 10-acre tract, are very coarse.

Because of the aggregation of subcatchments, each composite subcatchment was not representing any unique properties of ground cover as in the Chicago case. Some scheme had to be devised to compute the average physical properties, such as detention depth, infiltration rate, and Manning's coefficient, for the composite subcatchment. In this study, all the physical parameters, except Manning's coefficient, were calculated by a weighted average based on the percent perviousness of the area. For example, the detention depth is 1/16-inch for an impervious area and 1/4-inch for a pervious area (Ref. 5). The detention of a composite


N
SHADYSIDE RD.
RALWORTH RD.
ARGONNE DR.
TIVOLY AVE.
NORTHWOOD SHOPPING CTR.
PARKING
HAVERWOOD RD.
FENWICK AVE.
HILLEN RD.
Rain Gage #1
Rain Gage #2
Parshall Flume
Outfall
Creek
1
2
3
4
5
6
61
7
8
9
10
11
DRAINAGE AREA BOUNDARY
SUBCATCHMENT BOUNDARY
3 SUBCATCHMENT NUMBER

Source: L. S. Tucker, "Northwood Gaging Installation, Baltimore-Instrumentation and Data" (Ref. 10).

Figure 5-6. NORTHWOOD (BALTIMORE) DRAINAGE BASIN PLAN

subcatchment with x fraction impervious was calculated by:

$$\text{Average detention} = \frac{x}{16} + \frac{(1-x)}{4} \qquad (12)$$

In the case of Manning's coefficient n , a harmonic mean was taken

$$\frac{1}{n} = \frac{x}{0.014} + \frac{(1-x)}{0.35} \qquad (13)$$

where 0.014 and 0.35 are the Manning's coefficients for impervious and pervious areas respectively (Ref. 6).

The results of a simulation for the Northwood storm of August 1, 1965, are shown in Figure 5-7. For the purpose of comparison, the measured runoff is also plotted in the same figure. Again, the simulation is quite good, with accurate prediction of the peaks. The problem of slow recession is seen in the figure and is perhaps due to high values of Manning's coefficient being used for pervious areas.

It must be pointed out that all the necessary coefficients used in the simulation were estimated from Chicago data. The largest pipe diameter of the system is 4 feet.

## Additional Testing

While the discretization of the drainage basin into finer subcatchments is theoretically more correct, the coarsening of the system is desirable from the practical standpoint. However, there are problems associated with the gross approximation of the system by large subcatchments.

In the case of the Selby Street drainage basin in San Francisco, the average detention depth requirement for a gross subcatchment was found

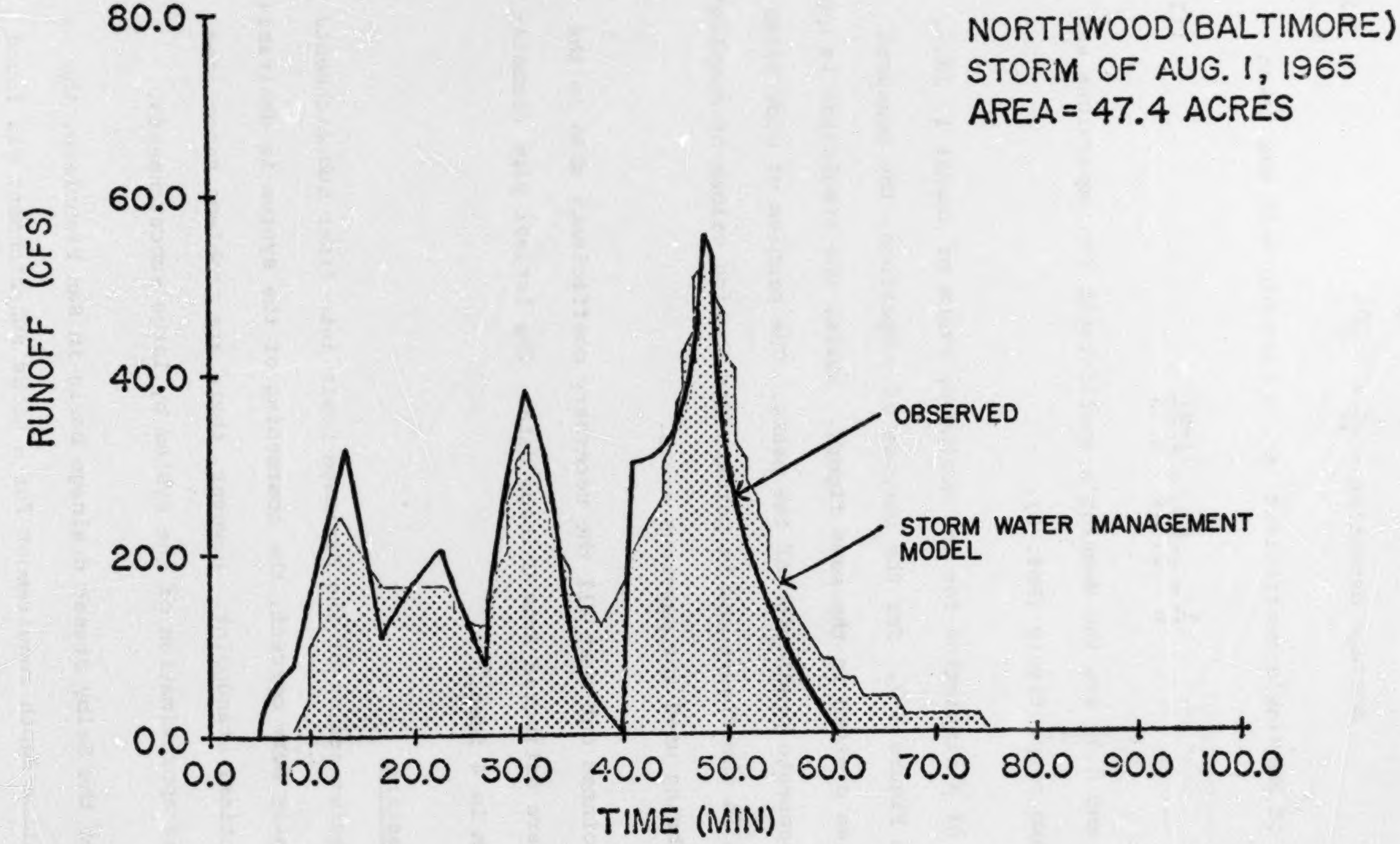


Figure 5-7. CALCULATED AND OBSERVED RUNOFF HYDROGRAPHS, NORTHWOOD (BALTIMORE)

to store the early low intensity rainfall. The rising limb of the hydrograph is therefore retarded greatly.

This problem was resolved by splitting each subcatchment into a pervious area and an impervious area according to the percent imperviousness of the ground cover. Since the impervious area requires small detention depth, it produces early runoff for the hydrograph. The use of "equivalent" detention depths and friction coefficients is avoided entirely.

The capability to split the area was incorporated into the computer program and tested with the data of Northwood (Baltimore). Figure 5-8 presents the result of the simulation for the 12-subcatchment system. The curve is noted to fit the observed data slightly better than the one shown in Figure 5-7. The reason it did not make more difference was that the early part of the rainfall intensity in this case was relatively high. This fact tends to reduce the importance of the detention depth.

Further analysis led to splitting each subcatchment into three parts:

1. Pervious
2. Impervious, with surface detention
3. Impervious, without surface detention.

This causes "instant" runoff from part of the system and is more in keeping with observed results. Use of this final version is described in the verification tests on Baker Street, San Francisco, in Volume II of this report.

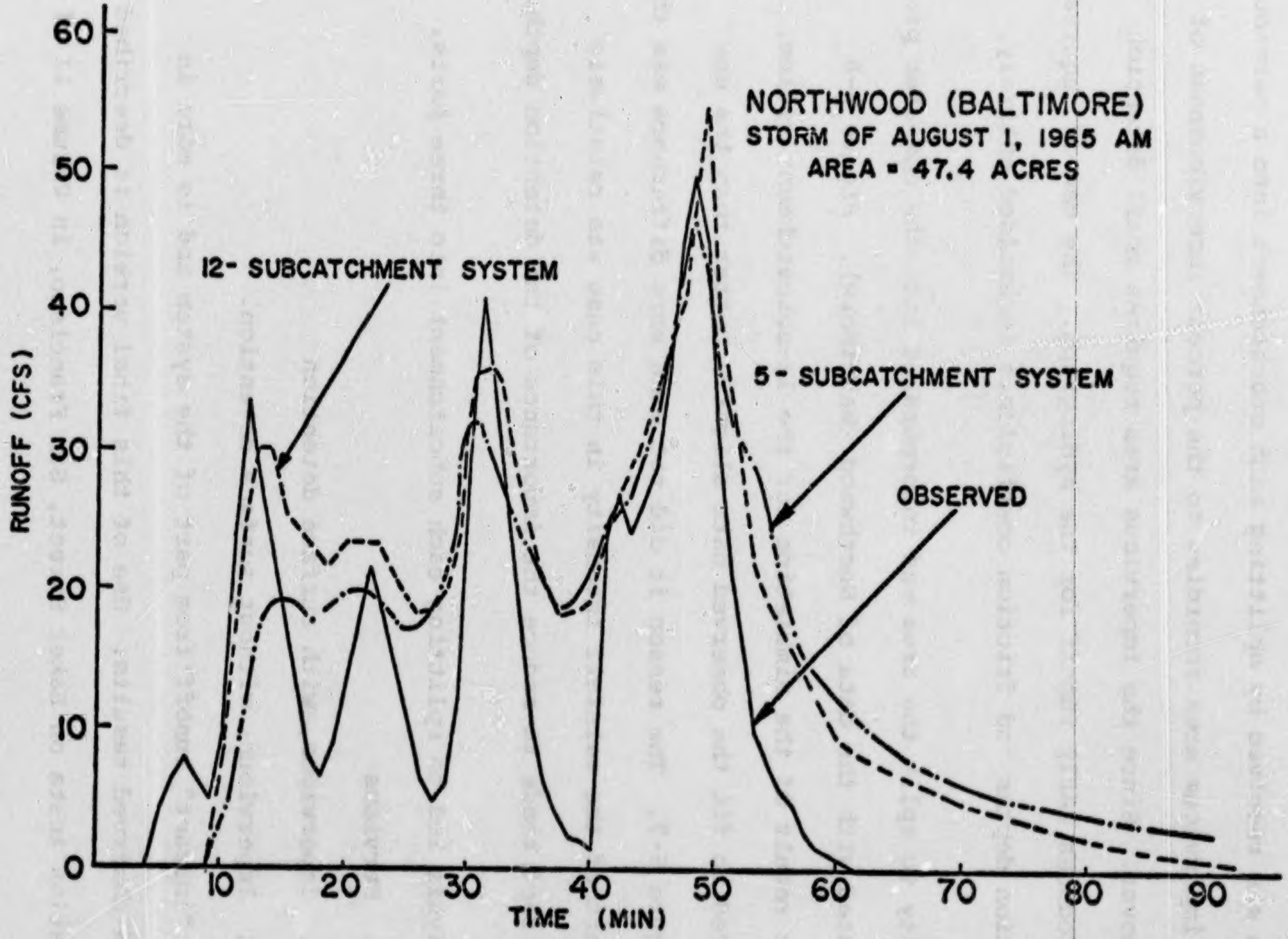


Figure 5-8. EFFECT OF COARSENING SUBCATCHMENT SYSTEM, NORTHWOOD (BALTIMORE)

Further coarsening of the subcatchment system was tested with the Northwood data. The 12-subcatchment system shown in Figure 5-6 was combined into a 5-subcatchment system. The result of the simulation is presented in Figure 5-8. As expected, the coarsening of the system reduces the accuracy of prediction. More experience with the model will be needed to ascertain the level of detail necessary for the accuracy desired in any specific application.

## CONCLUSION

The method described is only a near theoretical approximation of physical phenomena occurring in the prototype. The propagation of waves and the problem of varied flow overland and in the gutters are not treated rigorously in the method. However, the quasi-steady state approximation by Manning's equation is believed adequate for practical purposes in view of the fact that prototype conditions are not amenable to a theoretically more thorough analysis.

Aside from theoretical soundness, the validity of the method can best be demonstrated by its ability to reproduce the field data. Simulation results indicate that the verification is reasonably good. Moreover, all the parameters and coefficients utilized in the simulation were taken directly from published literature. No optimization procedure was used to adjust the coefficients for a better fit between the calculated and observed hydrograph.

The use of a discretized system to represent a drainage area increases the flexibility and latitude for the model to be adopted to other prototype conditions. The capability of the Surface Runoff Model to treat the pipe problem also enables more subcatchments to be aggregated into a larger tributary area. Subcatchments interconnected by pipes that lead to a common confluent point can be combined into an inlet drainage basin. The aggregation of subcatchments, however, should only be made to the extent where the inherent assumptions still hold. For example, the pipe routine uses the invert slope to calculate flow; therefore, the Surface Runoff Model should not be expanded to cover the larger size pipes where the backwater effect becomes important. It is believed that the Surface Runoff Model would be applicable for lateral sewers with pipe diameters up to 30 inches in most cases. Beyond that, the problem should be handled by the Transport Model which is described in Section 8 of this report.

# SECTION 6

## DRY WEATHER FLOW QUANTITY MODEL

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| Input Data | 81 |
| Methodology | 84 |
| TEST APPLICATIONS | 87 |
| CONCLUSION | 94 |

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# SECTION 6

## DRY WEATHER FLOW QUANTITY MODEL

In the context of the Storm Water Management Model, the quantity and quality of dry weather flow are important as they occur both prior to and during periods of storm runoff in combined sewers. At the onset of storm runoff, the initial volume in the sewer and the amount of settled and flowing pollutants establish initial conditions for flow and pollutant routing. This factor is especially important in assessing initial flushing of sewers by runoff. During runoff, DWF, although having lost its hydraulic significance, is important because of its relatively large pollutant concentration. Consequently, DWF serves as a significant weighting factor in mass balance relationships affecting pollutant concentrations.

In this study, separate modeling was undertaken for wastewater flow, wastewater quality, and groundwater infiltration. DWF quality, although estimated concurrently with quantity in the subroutine FILTH, is thus discussed separately in Section 12 of this volume. In addition, DWF quantity, as estimated in subroutine FILTH and defined in the following discussion, consists of only wastewater flow and excludes any estimate of infiltration. Although DWF in sewers actually consists of dry weather infiltration mixed with domestic, commercial, and industrial sewage, infiltration is modeled separately by subroutine INFIL and, as a result, is discussed separately in Section 7 of this volume. The following discussion therefore describes the development and testing of subroutine

FILTH as a predictive model for average wastewater flows from residential, commercial, and industrial urban areas.

## OBJECTIVES

Objectives in the development of subroutine FILTH were to predict accurately average wastewater flows and total pollutant loads. DWF's significance to the Storm Water Management Model was originally thought to be in establishing initial conditions for flow routing. However, comparisons of outfall hydrographs routed both with and without initialized DWF showed identical results after the first few minutes of overflow. Subsequent development has shown that DWF's primary significance lies in establishing initial quality conditions. Initial sediment load on sewer inverts and initial flow concentrations represent a relatively high percentage of initial pollutant overflow. Subsequent analyses showed also that hourly variation in average flow was required to describe DWF quality adequately.

## THE MODEL SUBROUTINE

The computer program developed to perform the DWF quantity computation is subroutine FILTH. This subroutine also computes the DWF quality. Subroutine FILTH is called from subroutine TRANS.

Daily and hourly flow variation for the study area are input along with flow quantities from process flows, if they are known. Information regarding total population, population density, land use, residential income, and home valuation, among other factors, are also required. The DWF quantity is then computed and passed back to subroutine TRANS.

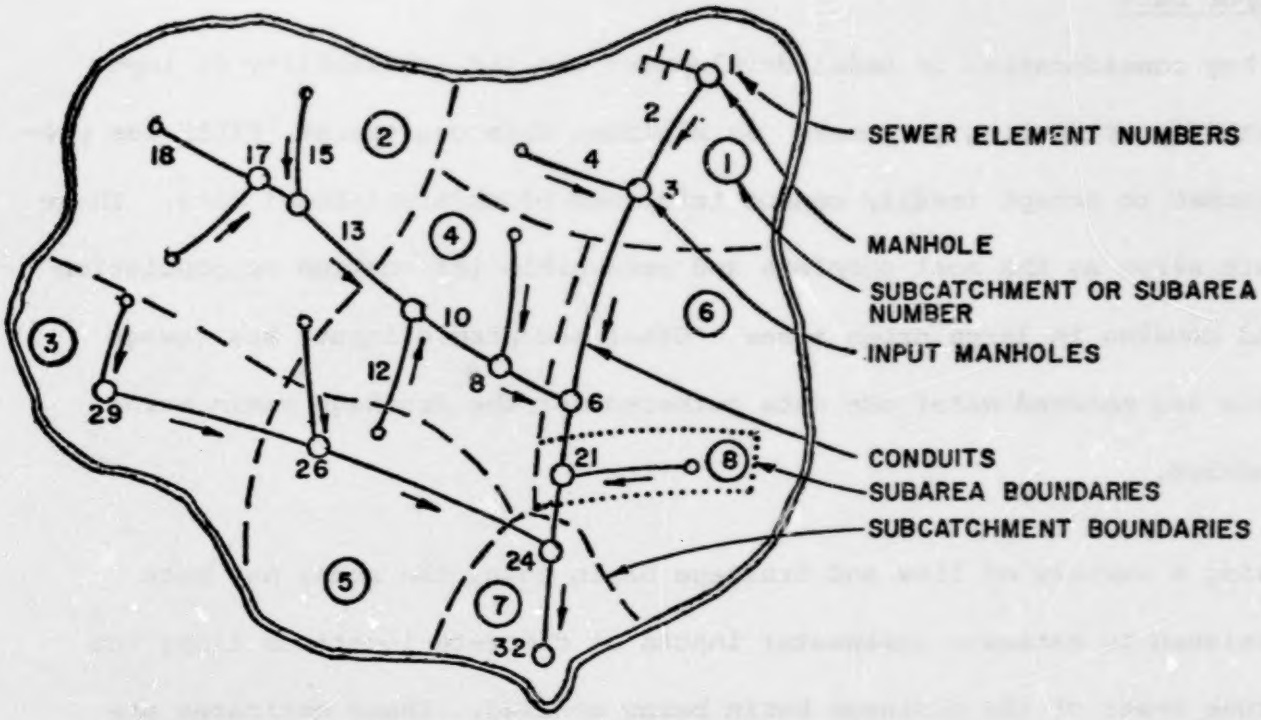
## THEORETICAL DEVELOPMENT

### Input Data

A key consideration in model development was the availability of input data for estimating purposes. To minimize this constraint, FILTH was programmed to accept readily census tract and block statistical data. These data serve as the most complete and accessible information on population and housing in large urban areas. Other acceptable inputs are sewage flow and metered water use data gathered for the drainage basin being modeled.

Using a variety of flow and drainage basin data, the model has been designed to estimate wastewater inputs at discrete locations along the trunk sewer of the drainage basin being modeled. These estimates are calculated from data describing drainage basin subsections (subcatchments) through which the trunk sewer passes. As shown in Figure 6-1, an input manhole near the center of each subcatchment is assumed to accept all sewage flow from that subcatchment. Sewage inputs from all subcatchments are then added to infiltration estimates from all pipe sections, and in the case of combined sewers, to runoff estimates from all inlet subcatchments. Noting the locations and timing of all inputs, the Transport Model (subroutine TRANS) then produces hydrographs at any specified location along the sewer.

Possible sources of DWF, other than for household wastewater, are listed in Table 6-1 (Ref. 1). Because of its highly unpredictable nature, discharge from swimming pools has been excluded as a source of DWF both


SEWER ELEMENT NUMBERS
MANHOLE
SUBCATCHMENT OR SUBAREA NUMBER
INPUT MANHOLES
CONDUITS
SUBAREA BOUNDARIES
SUBCATCHMENT BOUNDARIES
1
2
3
4
5
6
7
8
2
3
4
6
8
10
12
13
15
17
18
21
24
26
29
32

**Sewer and Subcatchment Data**

1. Manhole 32 is the most downstream point.
2. Subcatchments 1,2,3, and 4 are single-family residential areas, each 100 acres in size and each with water metering.
3. Subcatchments 5 and 7 are 220-acre industrial areas.
4. Subarea 6 is a 250-acre park.
5. Subarea 8 is a 50-acre commercial area.

Subareas 6 and 8 constitute a subcatchment draining to input manhole number 21.

**Resulting Data**

8 sewage estimates
KTNUM, total subcatchments and subareas in drainage basin = 8.
TOTA, total acres in drainage basin = 1,140.

| KNUM, subcatchment or subarea | INPUT, input manhole number | KLAND, land use category | ASUB, acres in subcatchment or subarea |
|---|---|---|---|
| 1 | 3 | 1 | 100 |
| 2 | 17 | 1 | 100 |
| 3 | 29 | 1 | 100 |
| 4 | 8 | 1 | 100 |
| 5 | 26 | 4 | 220 |
| 6 | 21 | 5 | 250 |
| 7 | 24 | 4 | 220 |
| 8 | 21 | 3 | 50 |

Figure 6-1. DETERMINATION OF SUBCATCHMENT AND IDENTIFICATION DATA TO ESTIMATE SEWAGE AT 8 POINTS

Table 6-1. ALLOWABLE NUMBER OF SEWER CONNECTIONS BY CITY SIZE AND GEOGRAPHIC LOCATION

| | Connections Are Allowed for the discharge of | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Swimming Pools | | Foundation Drains | | Roof Leaders | | Sump Pump | | Cooling Water | | Treated Industrial Process Water | | Untreated Industrial Process Water | |
| | Yes | No | Yes | No | Yes | No | Yes | No | Yes | No | Yes | No | Yes | No |
| **Combined Sewers** | | | | | | | | | | | | | | |
| City size | | | | | | | | | | | | | | |
| 0-10,000 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| 10,001-25,000 | 8 | 4 | 9 | 3 | 8 | 4 | 10 | 2 | 8 | 3 | 6 | 4 | 5 | 4 |
| 25,001-100,000 | 9 | 3 | 10 | 2 | 5 | 7 | 11 | 1 | 9 | 2 | 8 | 1 | 7 | 2 |
| 100,001-1,000,000 | 13 | 2 | 12 | 3 | 12 | 3 | 14 | 1 | 15 | -- | 12 | 1 | 9 | 3 |
| Over 1,000,000 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| Geographic location | | | | | | | | | | | | | | |
| East | 15 | 5 | 15 | 5 | 14 | 6 | 17 | 3 | 15 | 3 | 13 | 2 | 10 | 5 |
| South | 1 | -- | 2 | -- | 1 | 1 | 2 | -- | 2 | -- | 2 | -- | 1 | 1 |
| Midwest | 7 | 4 | 9 | 2 | 5 | 6 | 10 | 1 | 10 | 1 | 7 | 2 | 6 | 2 |
| Southwest | 1 | -- | 1 | -- | 1 | -- | 1 | -- | 1 | -- | 1 | -- | 1 | -- |
| West | 5 | -- | 4 | 1 | 4 | 1 | 5 | -- | 4 | 1 | 3 | 2 | 3 | 1 |
| Total | 29 | 9 | 31 | 8 | 25 | 14 | 35 | 4 | 32 | 5 | 26 | 6 | 21 | 9 |
| **Storm Sewers** | | | | | | | | | | | | | | |
| City size | | | | | | | | | | | | | | |
| 0-10,000 | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- | -- |
| 10,001-25,000 | 14 | 5 | 19 | 2 | 18 | 2 | 15 | 5 | 16 | 2 | 6 | 7 | 5 | 6 |
| 25,001-100,000 | 27 | 6 | 32 | 2 | 35 | 1 | 26 | 4 | 27 | 6 | 15 | 13 | 4 | 19 |
| 100,001-1,000,000 | 21 | 5 | 23 | 3 | 25 | 2 | 23 | 5 | 28 | 1 | 16 | 7 | 8 | 15 |
| Over 1,000,000 | 1 | -- | 1 | -- | 1 | -- | 1 | 1 | 1 | -- | -- | 1 | -- | -- |

Source: APWA, "Water Pollution Aspects of Urban Runoff," January 1969, WP-20-15 (Ref. 1).

prior to and during runoff. Roof leaders, foundation drains, and sump pump connections were not considered in the realm of DWF to the extent that they provide alternate paths for surface runoff and, therefore, affect the travel time of runoff to storm and combined sewers. Industrial flows are input to the Transport Model via FILTH.

## Methodology

Model development relied heavily on major research programs conducted at the Johns Hopkins University (Ref. 2). These efforts were concerned with quantitative formulations of residential and commercial water usage patterns based on extensive programs of meter installation and data review. The residential work clearly exceeded the scope, magnitude, and depth of consideration of any work preceding it. It was national in scope and utilized advanced statistical techniques for data evaluation. The commercial usage study, on the other hand, was centered about Baltimore, Maryland. Because of its well organized approach and thoroughness, the commercial water use study has validity for use on a national scale, as well. Additional estimates of water use were obtained from a computerized water use study by Hittman and Associates from Columbia, Maryland (Ref. 3), and a summary of water use in manufacturing from the U. S. Census Bureau (Ref. 4).

Based upon the previously mentioned objectives and constraints, the structure and logic of FILTH was developed to read and edit input data, make necessary flow estimates, and coordinate flow estimates as inputs to the Transport Model (subroutine TRANS). FILTH was structured to give priority to sewage flow and metered water use data as predictors of wastewater flow. If estimates were necessary for commercial and industrial subareas, water use coefficients provided in the "User's Manual" (subroutine FILTH) provided the necessary input (Refs. 2,3,5). Flow estimates for residential areas are made within the computer program using water use estimating equations developed in the Johns Hopkins Residential Water Use Study.

Estimating equations for residential water use were taken from the work done by Linaweaver. From analysis of water use throughout the country, Linaweaver stated that the principal factor influencing total annual water use in any residential area is the total number of homes. Three other major factors are: economic level of the consumer, climate, and whether consumers are metered or on a flat rate basis.

Income level of the consumer influences domestic water for two primary reasons. According to Linaweaver's investigations, consumers in a high-valued area have more water-using appliances. As a result, domestic use is likely to increase because the average time between uses decreases while the average duration per use increases. To verify this assumption, Linaweaver analyzed water use data from 29 areas and developed the following regression equation:

$$Q_{d/a} = 178 + 3.28v \qquad (1)$$

where $Q_{d/a}$ = Average domestic use per dwelling unit (gpd)

$v$ = Average assessed valuation of property (thousands of dollars)

Subsequent analysis of residential water demand by Howe and Linaweaver (Ref. 6) resulted in the following equation:

$$q_{a,d} = f(v, a, d_p, k, p_w) \qquad (2)$$

where $q_{a,d}$ = Average annual quantity demanded for domestic purposes (gpd per dwelling unit)

$v$ = Market value of the dwelling unit (thousands of dollars)

$a$ = Age of the dwelling unit (yr)

$d_p$ = Number of persons per dwelling unit

$k$ = Average water pressure (psi)

$p_w$ = Sum of water and sewage charges that vary with water use, evaluated at the block rate applicable to the average domestic use in each drainage basin

After extensive analysis of water use data from 21 eastern and western drainage basins, variabilities in average water use due to the existence of public sewers, flat rate pricing, and apartments were statistically blocked out. Regression of average daily water use on the remaining variables yielded the following linear equations for the type of residential areas specified.

Metered with public sewer:

$$q_{a,d} = 206 + 3.47v - 1.30\ p_w \qquad (3)$$

Flat rate and apartments with public sewer:

$$q_{a,d} = 28.9 + 4.39v + 33.6d_p \qquad (4)$$

Metered with septic tanks:

$$q_{a,d} = 30.2 + 39.5d_p \qquad (5)$$

## TEST APPLICATIONS

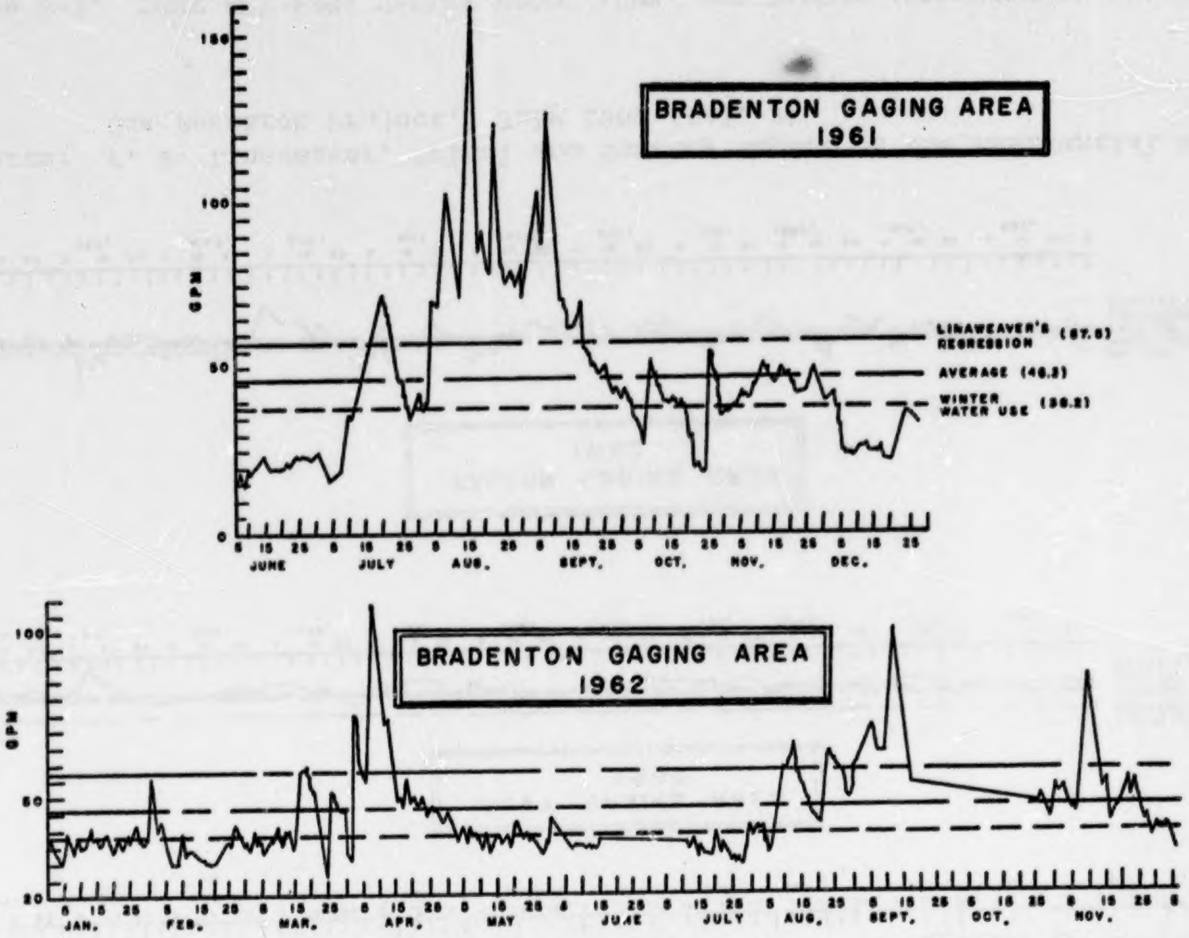
Preliminary testing of FILTH was accomplished through the use of data from the Johns Hopkins University Residential Sewerage Research Project. A summary of the results of this testing is given in Table 6-2 and Figures 6-2 to 6-6. For subcatchments less than 130 acres in size, it can be seen that deviations may be as high as 108 percent and average nearly 40 percent of recorded flow. However, application to one 3,400-acre drainage basin in San Francisco resulted in underestimation of sewage flow by only 12 percent. Similar accuracy was noted in the verification applications of the Storm Water Management Model as described in Volume II of this report. The significance of the seemingly large variability of estimates in small upstream subcatchments lies in the establishment of initial flow in the sewers prior to the occurrence of runoff. Extreme upstream branches of the sewer may have dry weather flows that are somewhat above or below actual conditions. These discrepancies are soon dampened, however, as flows accumulate downstream and have been shown to be less than 12 percent at the outfall.

During testing of FILTH, it was noted that recorded dry weather sewage flows and quality exhibited definite variation depending on the time of the day and day of the week. These variations were accounted for in FILTH by correcting estimated flow by hourly and daily flow correction factors. These factors can be determined from historical flow records as described in the "User's Manual."

Table 6-2. ACTUAL AND ESTIMATED DAILY AVERAGE RESIDENTIAL SANITARY SEWER FLOWS TO TEST VALIDITY OF SUBROUTINE FILTH

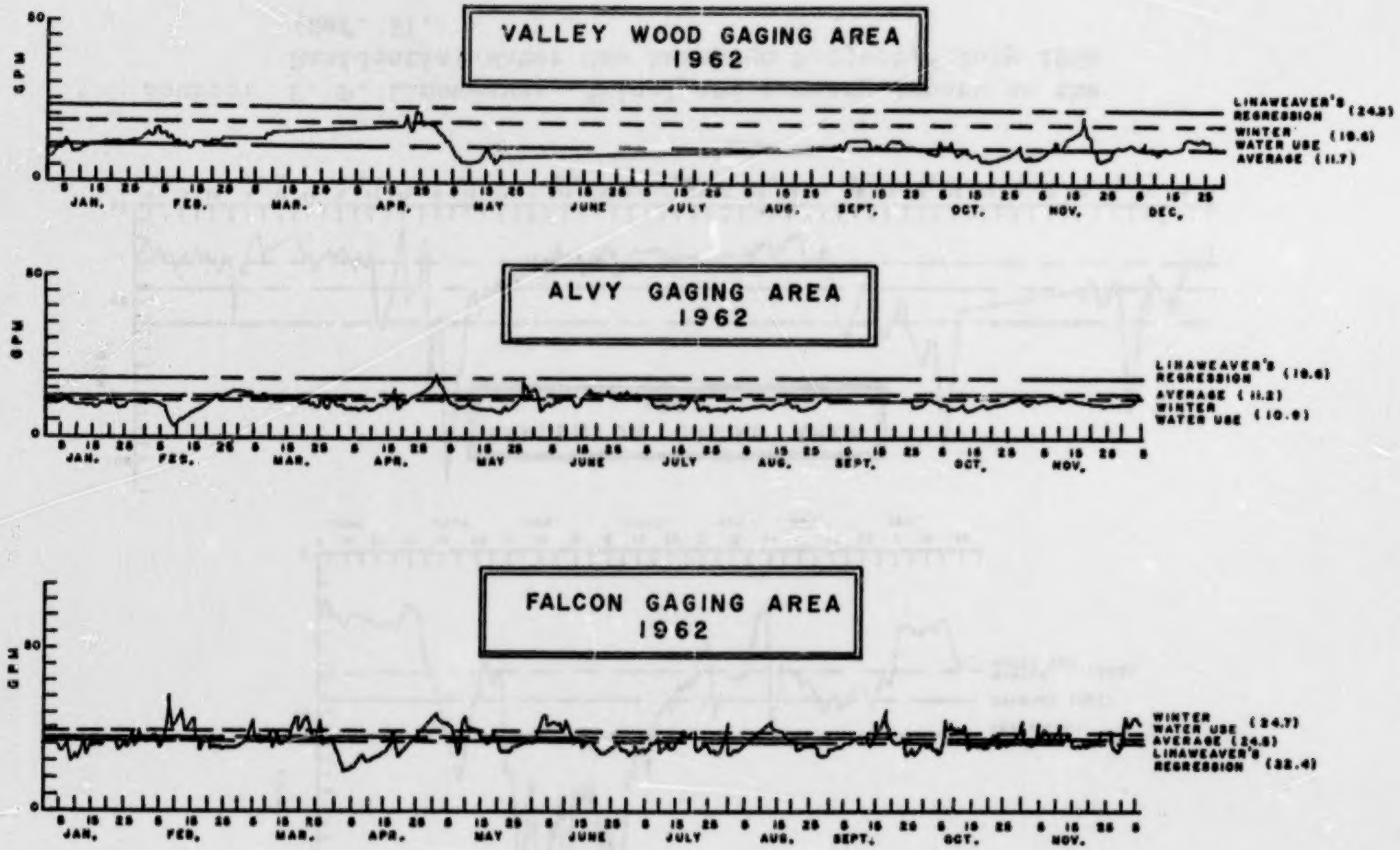
| Gaging Area | Area, acres | 1 Measured* Sewage Flow, gpm | 2 Estimated Water Use, gpm | Deviation 2-1 | | 3 Measured Water Use, gpm | Deviation 3-1 | |
|---|---|---|---|---|---|---|---|---|
| | | | | gpm | percent | | gpm | percent |
| Bradenton (Florida) | 130 | 46.32 | 57.50 | +11.18 | 24.1% | 38.20 | - 8.12 | 17.5% |
| Falcon (California) | 34 | 24.50 | 22.40 | - 2.10 | 8.6 | 24.70 | + 0.20 | 8.8 |
| Nutwood (Maryland) | 112 | 87.50 | 85.92 | - 1.58 | 1.8 | 95.54 | + 8.04 | 9.1 |
| Valley Wood (California) | 63 | 11.66 | 24.34 | +12.68 | 108.7 | 19.59 | + 7.93 | 68.0 |
| Springfield (Missouri) | 89 | 37.59 | 49.10 | +11.51 | 30.6 | 28.10 | - 9.49 | 25.2 |
| Alvy (California) | 12 | 11.18 | 19.64 | + 8.46 | 75.7 | 10.86 | - 0.32 | 2.8 |
| Pine Valley (Maryland) (corrected) | 47 | 45.20 | 25.18 | -20.02 | 44.3 | 31.58 | -13.62 | 30.1 |

*Except for Pine Valley (corrected), flow includes groundwater infiltration and storm water penetration.



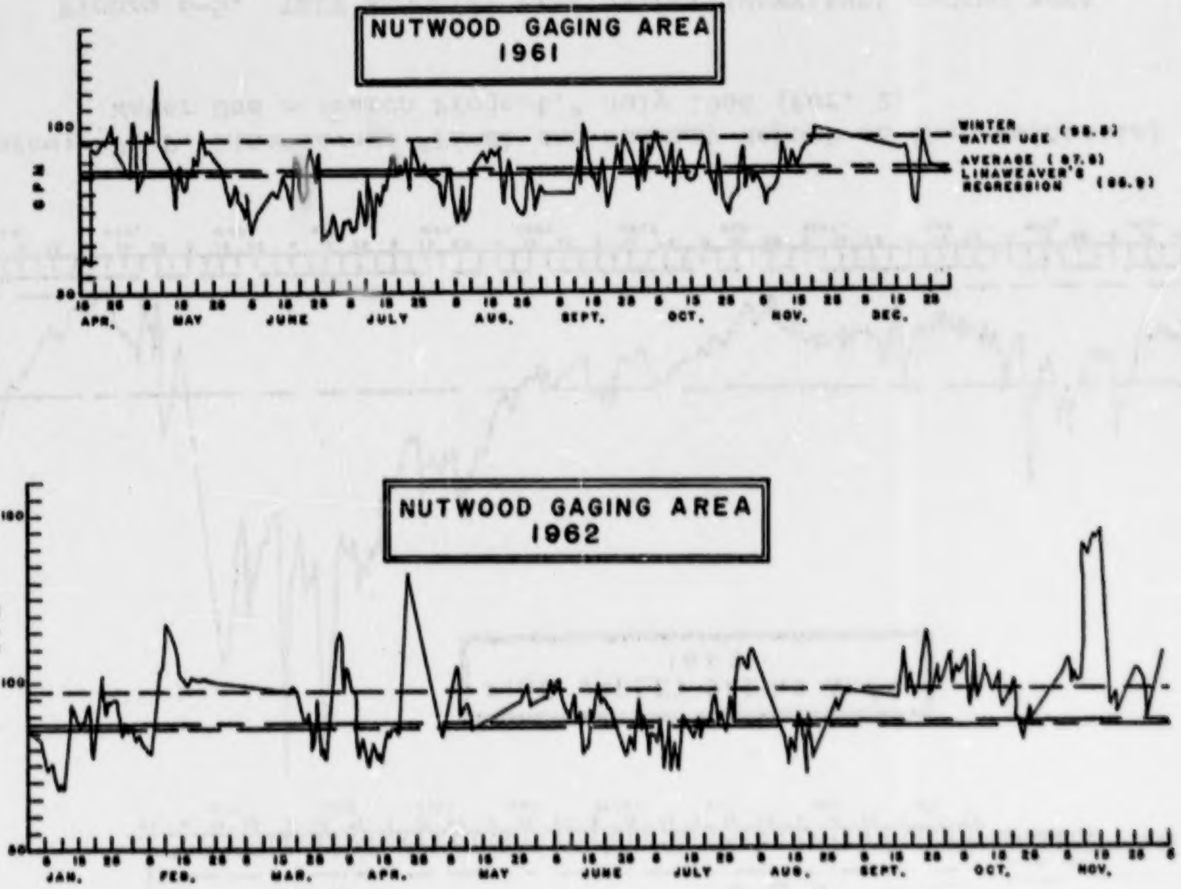
Source: F. P. Linaweaver, "Final and Summary Report on the Residential Water Use Research Project," July 1966 (Ref. 2).

Figure 6-2. TEST RESULTS, BRADENTON (FLORIDA) GAGING AREA



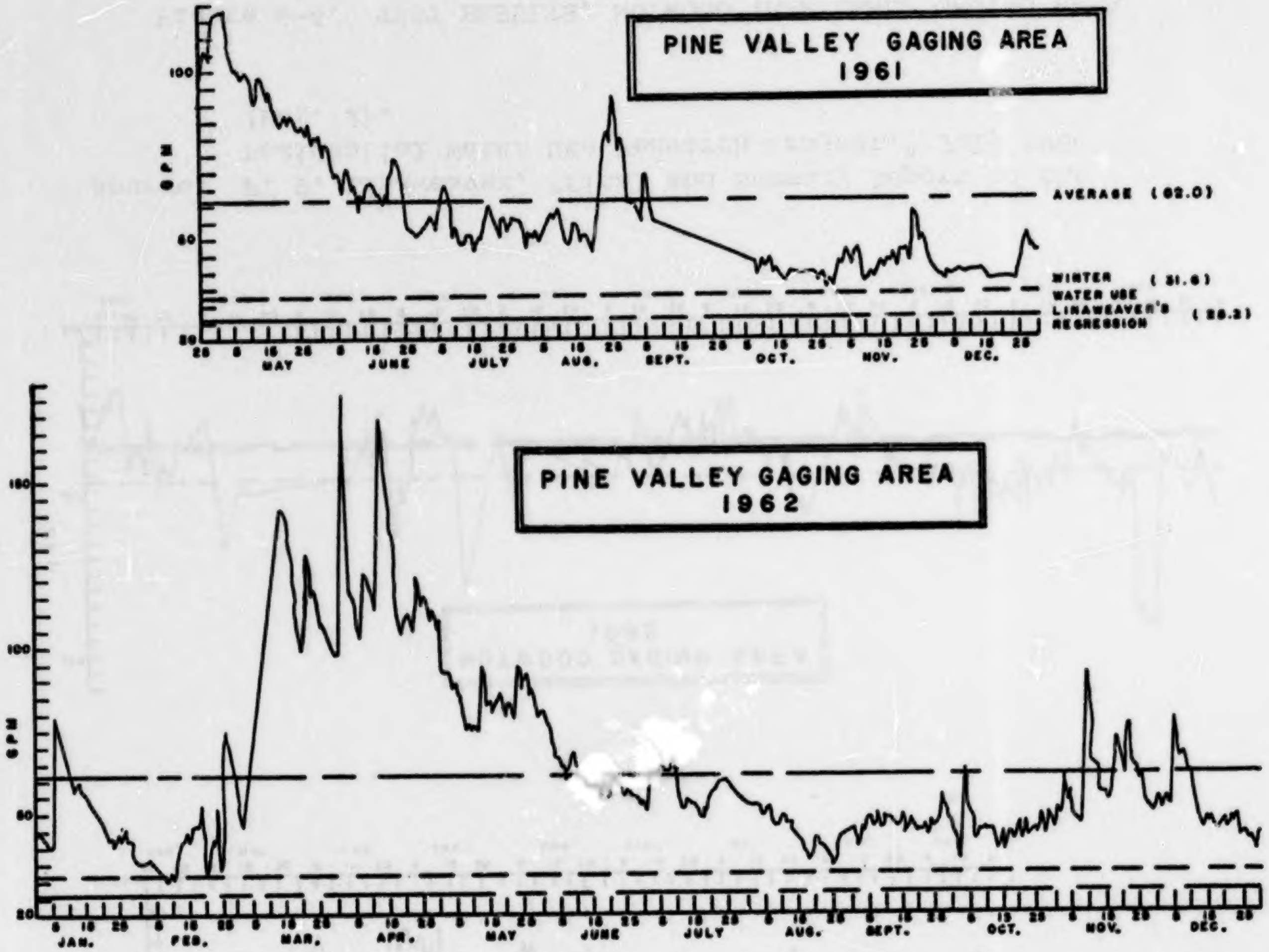
Source: F. P. Linaweaver, "Final and Summary Report on the Residential Water Use Research Project," July 1966 (Ref. 2).

Figure 6-3. TEST RESULTS, VALLEY WOOD, ALVY, AND FALCON (CALIFORNIA) GAGING AREAS



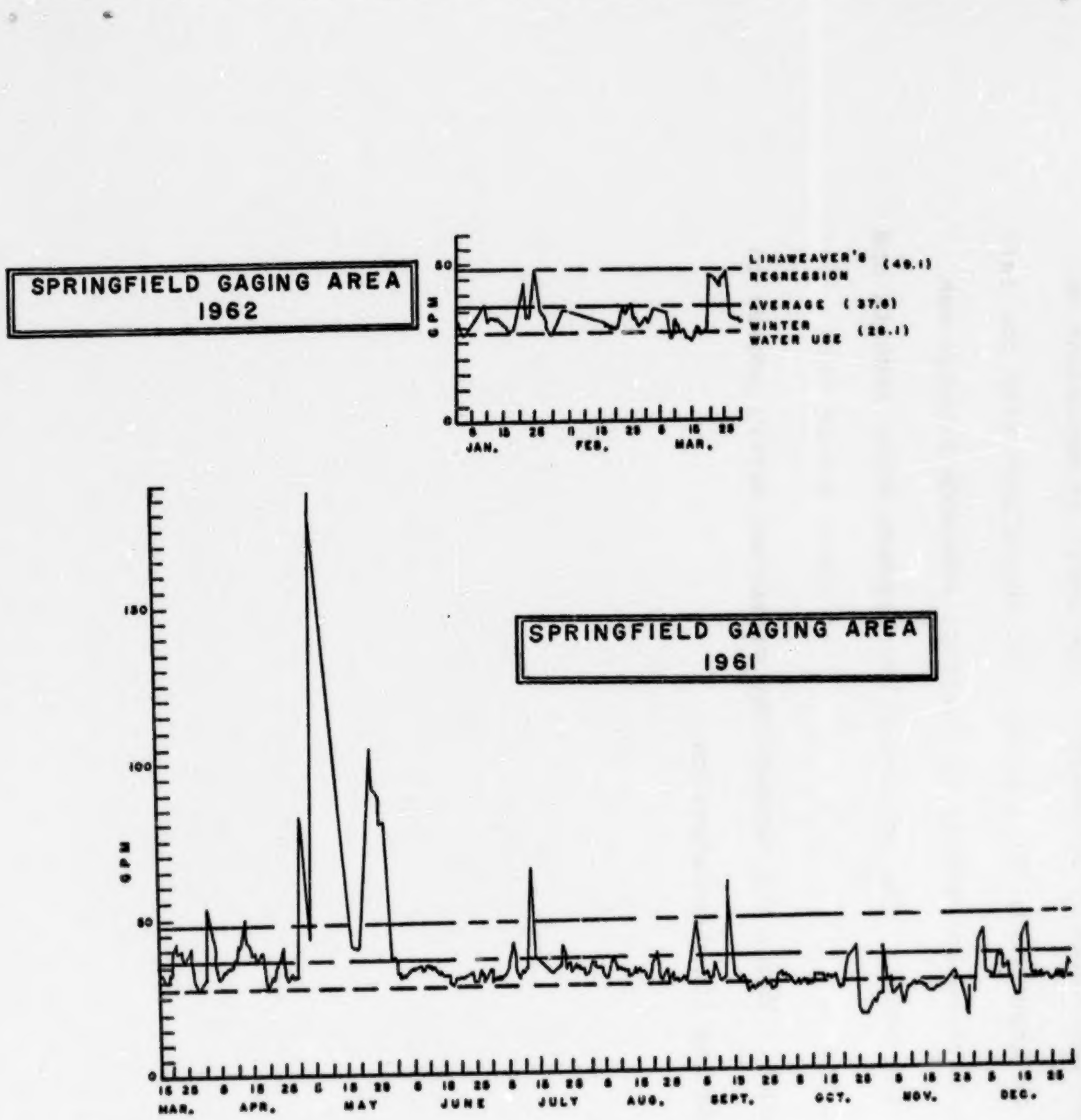
Source: F. P. Linaweaver, "Final and Summary Report on the Residential Water Use Research Project," July 1966 (Ref. 2).

Figure 6-4. TEST RESULTS, NUTWOOD (MARYLAND) GAGING AREA



Source: F. P. Linaweaver, "Final and Summary Report on the Residential Water Use Research Project," July 1966 (Ref. 2)

Figure 6-5. TEST RESULTS, PINE VALLEY (MARYLAND) GAGING AREA



Source: F. P. Linaweaver, "Final and Summary Report on the Residential Water Use Research Project," July 1966 (Ref. 2)

Figure 6-6. TEST RESULTS, SPRINGFIELD (MISSOURI) GAGING AREA

## CONCLUSION

The Dry Weather Flow model was developed to represent DWF quantity and quality for periods prior to and during storm runoff in the physical sewer system being studied. Using a regression analysis concept similar to the research accomplished by Linaweaver, the model is dependent on the availability of input data for a predictive comparison with the real world. The Johns Hopkins University Residential Research Project was used for testing the model. The percent error between model results and recorded flows at the outfall averaged to a reasonable amount when validating the model. Thus, the methodology presented herein proved to be satisfactory during its demonstration.

## SECTION 7

## INFILTRATION MODEL

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## SECTION 7

## INFILTRATION MODEL

In order to arrive at a model to determine the relative contribution of infiltration to flow and quality in combined sewers, two primary assumptions were made. It was assumed that a model could be developed to reflect the availability, movement, and entry of moisture into a subsurface conduit. The effect of infiltration upon quality was assumed negligible except for some dilution.

The present state of the art was such that actual flow measurements of infiltration for comparison to predictions were unavailable. This constraint was overcome by using minimum and average daily sewage flow data recorded on the Johns Hopkins University Residential Sewerage Research Project (Ref. 1). An analysis of the Infiltration model is presented in the remainder of this section.

### OBJECTIVES

The incorporation of the Infiltration model, subroutine INFIL, into the overall Storm Water Management Model and its adaptability to local variations and characteristics were of prime concern. Thus, in the development of INFIL, the objectives were:

1. To predict accurately infiltration into a given sewer system on the basis of existing local information about the sewer, its surrounding soil and groundwater, precipitation, and other climatological data.

2. To apportion the predicted infiltration at discrete locations along the given trunk sewers prior to flow routing.

## THE MODEL SUBROUTINE

The computer program developed to perform the infiltration computation is subroutine INFIL.

Subroutine INFIL is called from TRANS to compute the infiltration from groundwater, antecedent precipitation, and ice and frost melting. The infiltration is then combined with the DWF quantity computed in FILTH to yield the base flow in the sewer.

## THEORETICAL DEVELOPMENT

A typical urban drainage basin in which infiltration might be estimated for use by the Storm Water Management Model is shown in Figure 7-1.

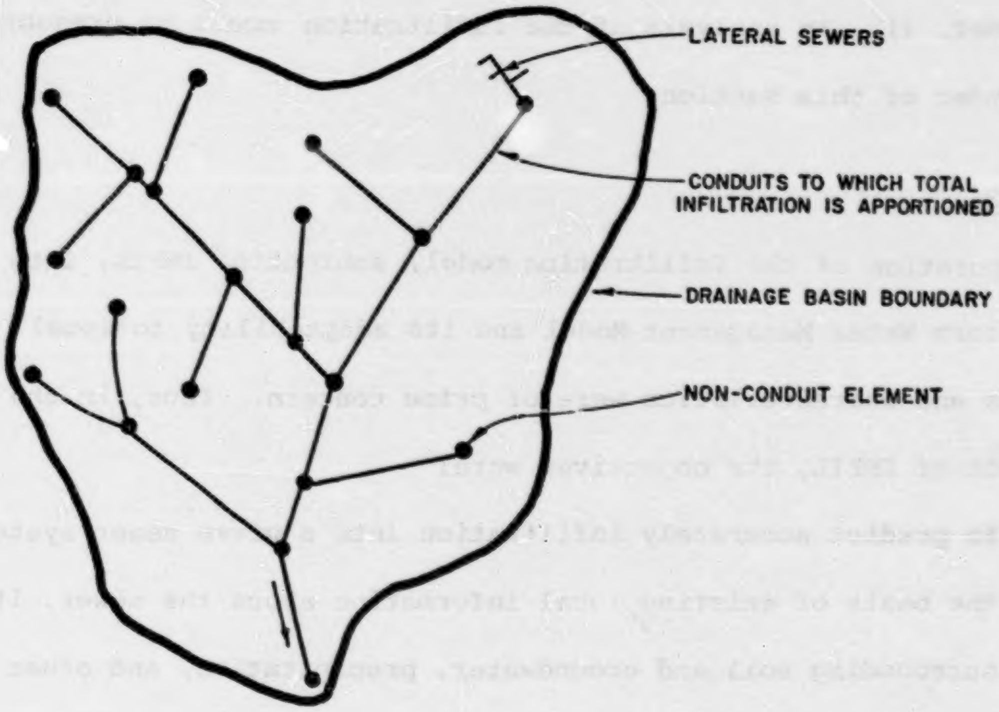

LATERAL SEWERS
CONDUITS TO WHICH TOTAL INFILTRATION IS APPORTIONED
DRAINAGE BASIN BOUNDARY
NON-CONDUIT ELEMENT

Figure 7-1. TYPICAL DRAINAGE BASIN IN WHICH INFILTRATION IS TO BE ESTIMATED

In the preliminary stages of model development consideration was given to the use of existing mathematical relationships representing flow through porous media. The complexity and inflexibility of these relationships (Ref. 2), together with the stochastic nature of the real problem, led to the selection of empirical methods.

Initial attempts were made to develop a general predictive equation of nationwide applicability for infiltration, as presented in Appendix A. The equation incorporated an antecedent precipitation index, pipe diameter, pipe length, and factors denoting soil type and joint material. However, it failed to account for abrupt flow changes and exhibited trends that only approximated local conditions and measurements. This lack of sensitivity to local conditions was further accentuated upon comparison with infiltration measurements taken by M&E in a study for the EPA on storm water problems and control in sanitary sewers in a six-city area along the east shore of San Francisco Bay (Ref. 3). The results of this comparison, however, led to the development of the present model.

Input Data

The availability of input data, therefore, was the key factor in developing the model. An analysis of variance on time-dependent variables and minimum flows was conducted on each of seven study areas from the Johns Hopkins study after obtaining supplementary climatological and geological information (Ref. 4). Data manipulation went as follows:

1. Determination of the existence of regression using the following recorded data:

    a. Precipitation

   b. Temperature

   c. Relative humidity

   d. Study area sewer descriptive parameters.

2. Elimination of variables accounting for the least amount of variance.

3. Transformation of remaining variables to obtain better correlation and regression as follows:

   a. Use of time delays from one to six days on rainfall

   b. Use of precipitation index to indicate soil moisture conditions.

From the above data manipulations, only precipitation significantly accounted for minimum flow variation in each study area. However, groundwater level was considered in the model when it occurred above the sewer invert, in which case the sewer was submerged and all infiltration was assumed due to this condition (Ref. 5). Data describing pipe size and the number of joints were retained to reflect entry of infiltration into conduits (Ref. 6). Infiltration into manholes was considered negligible, therefore elminating any need for manhole infiltration data.

Methodology

For analytical purposes, infiltration includes (1) moisture from miscellaneous sources causing a base dry weather inflow, (2) frozen residual moisture, (3) antecedent precipitation, and (4) high groundwater. Based upon this assumption, infiltration was defined as:

$$\mathrm{QINF} = \begin{cases} \mathrm{DINFIL + RINFIL + SINFIL} \\ \text{or} \\ \text{GINFIL for high groundwater table} \end{cases} \qquad (1)$$

where QINF = Total infiltration

DINFIL = Dry weather infiltration

RINFIL = Wet weather infiltration - DINFIL

SINFIL = Melting residual ice and snow infiltration

GINFIL = Groundwater infiltration

The cumulative effect of the first three sources can be seen in Figure 7-2 which shows total infiltration as the sum of dry weather infiltration, wet weather infiltration, and melting residual ice and frost infiltration. However, in cases where the groundwater table occurs above the sewer invert, it was assumed that groundwater alone would be the dominant source of infiltration.

High Groundwater Table (GINFIL). For locations and times of the year that cause the groundwater table to be above the sewer invert, groundwater infiltration, GINFIL, supersedes any notation of DINFIL, RINFIL, and SINFIL, as shown in Eq. 1. GINFIL is then determined from historical sewer flow data, by inspection or regression analysis. Regression analysis involves the determination of the coefficients in the following equation:

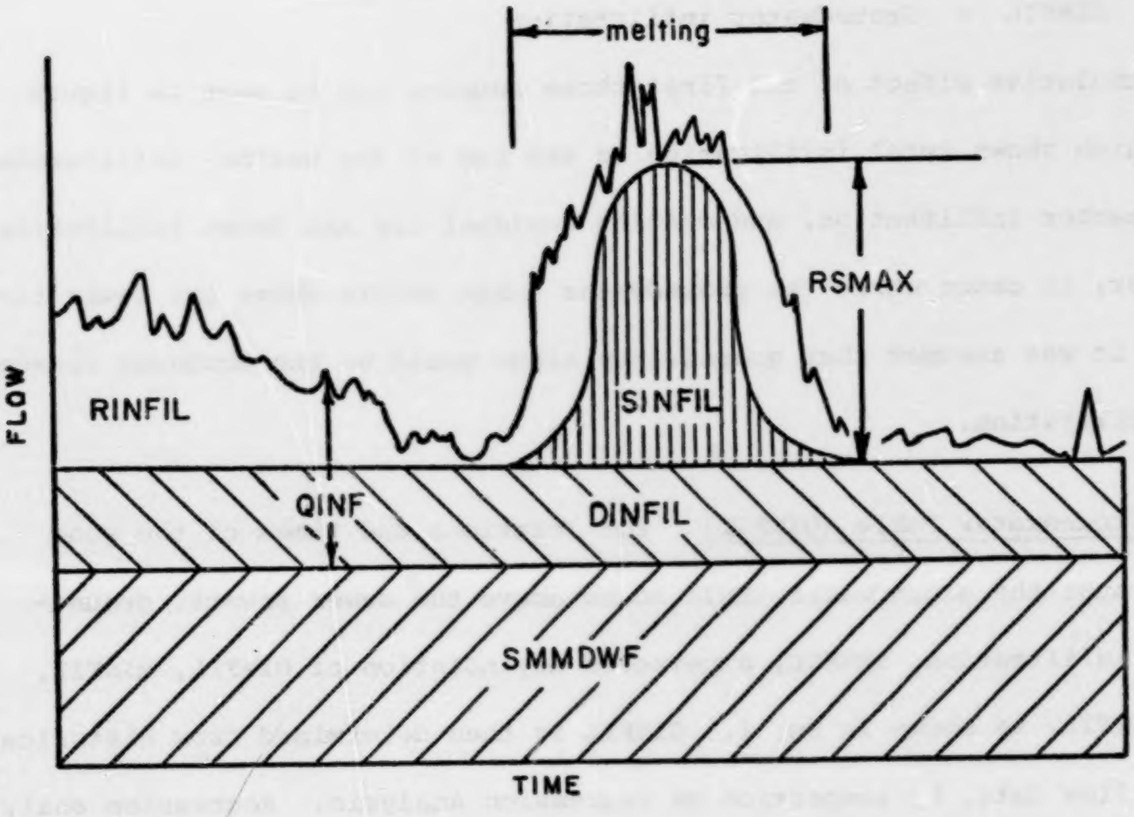
$$\text{GINFIL} = \sum_{i=1}^{n} (\text{BETA}(i))\,(\text{GWHD})^{\alpha_i} \qquad (2)$$

where GINFIL = Groundwater infiltration

GWHD = Groundwater table elevation above sewer invert (ft)

BETA(i) = Coefficient for ith term in equation

$\alpha_i$ = Power to which GWHD is raised in ith term, e.g., 0, 1, 2, 1/2 for term i = 0, 1, 2, and 3, respectively


melting
RSMAX
FLOW
RINFIL
SINFIL
QINF
DINFIL
SMMDWF
TIME

QINF = Total infiltration
DINFIL = Dry weather infiltration
RINFIL = Wet weather infiltration
SINFIL = Melting residual ice and snow infiltration
RSMAX = Residual moisture peak contribution
SMMDWF = Accounted for sewage flow

Figure 7-2. COMPONENTS OF INFILTRATION

**Estimation of Antecedent Precipitation (RINFIL).** Using available precipitation and average sewer flow data not affected by melting (Ref. 1), RINFIL was estimated using the following linear relationship:

$$RINFIL = ALF + (ALF_0)(RN_0) + (ALF_1)(RN_1) + \ldots + (ALF_9)(RN_9) \qquad (3)$$

where $RINFIL$ = SWFLOW - DINFIL - SMMDWF

$ALF_N$ = Coefficient to rainfall for N days prior to estimate, N = 0, 1, ..., 9

$DINFIL$ = Average dry weather flow minus the accountable sanitary flow

$RN_N$ = Precipitation on N days prior to estimate (in.), N = 0, 1, ..., 9

$SWFLOW$ = Daily average sewer flow excluding surface runoff (gpm)

$SMMDWF$ = Accounted for sewage flow (gpm)

In establishing this relationship for RINFIL, the characteristics of the local soil conditions are being taken into consideration indirectly. The uniqueness of this estimate and the need for establishing localized relationships is thus indicated.

**Estimation of Residual Melting Ice and Frost Infiltration (SINFIL).** The length of the melting period as well as the rate of melting was calculated by using published data (Ref. 7) in the form of degree-days as an index. Degree-day (NDD) is a unit based upon temperature difference and time. Based on observed relationships between the degree-days and melting periods in various geographical locations, the value of 750 degree-days was selected as the indicator of the point in time at which residual ice and snow would start accumulating and also as an indicator of the onset of the melting period.

As shown in Figure 7-3, the portion of the curve falling above 750 degree-days corresponds to the period of accumulation. The beginning of melting, MLTBE, is taken as the day on which NDD drops below 750. This melting period is assumed to end at that point in time (MLTEN) in which the area representing accumulated precipitation, $A_1$, equals the area $A_2$ during the melting period. Therefore, residual melting ice and frost infiltration (SINFIL) is determined by the following equation:

$$\mathrm{SINFIL} = \begin{cases} \mathrm{RSMAX} \left\{ \mathrm{Sin} \left[ 180 \left( \dfrac{(\mathrm{NDYUD} - \mathrm{MLTBE})}{(\mathrm{MLTEN} - \mathrm{MLTBE})} \right) \right] \right\} \\ \text{0.0 if the day under study is not} \\ \text{in the melting period or if NDD} \\ \text{never exceeds 750} \end{cases} \tag{4}$$

where SINFIL = Melting residual ice and snow infiltration

MLTBE = Day on which melting period begins

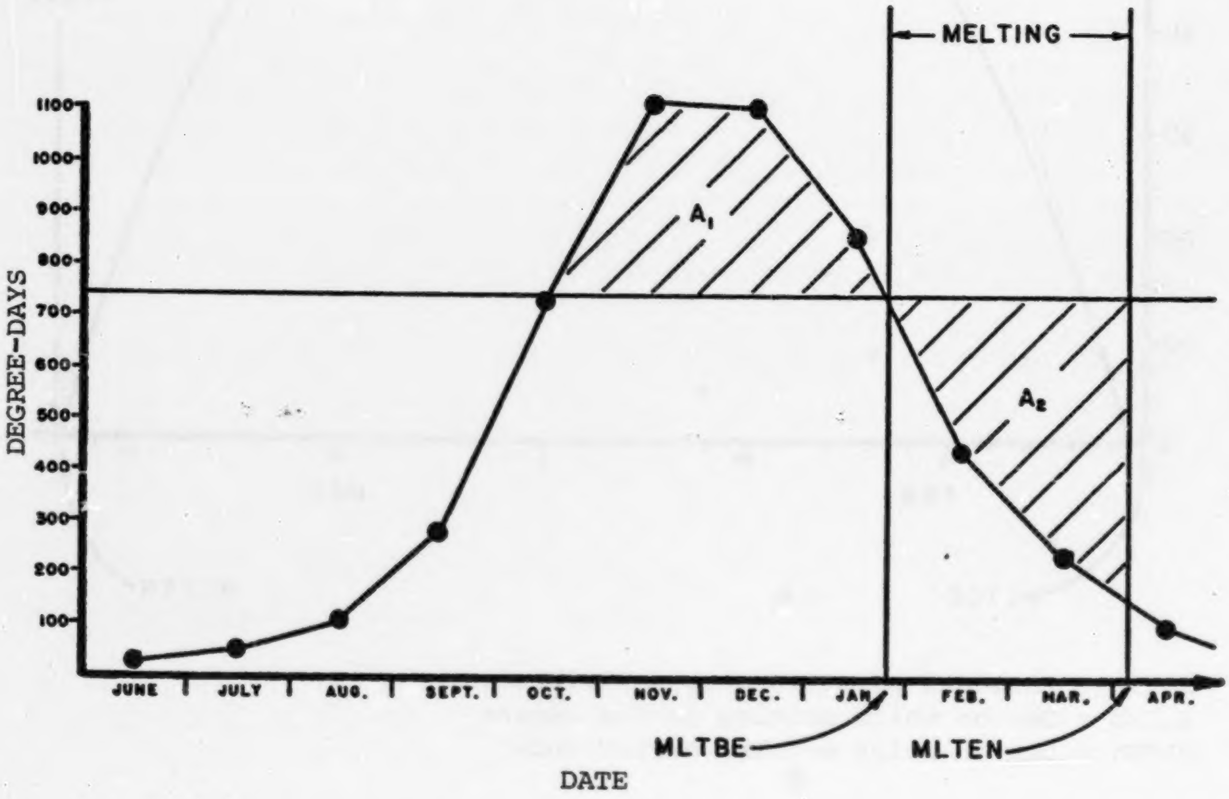
MLTEN = Day on which melting period ends

NDYUD = Day on which infiltration estimate is desired

RSMAX = Residual moisture peak contribution

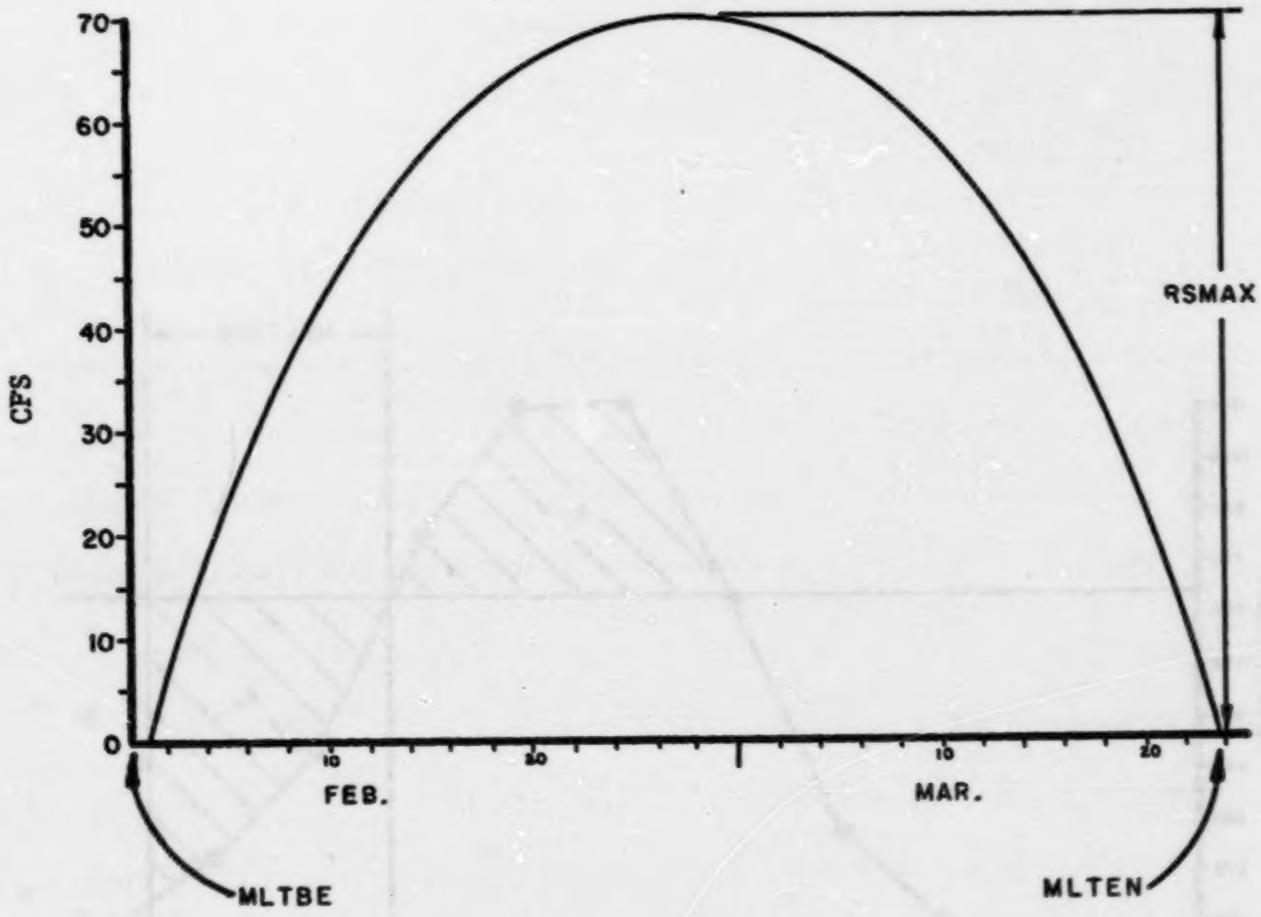
In the absence of evidence to the contrary, the rate of melting is assumed to be sinusoidal. This is shown in Figure 7-4 where RSMAX represents the maximum contribution from residual moisture and is determined from previous gaging of the study area or from local estimates.

Apportionment of Infiltration. Once an estimate of local infiltration, QINFIL, has been obtained, this flow must be apportioned throughout the designated study area. The criterion chosen for apportionment is an



MLTBE = Day on which melting period begins
MLTEN = Day on which melting period ends

Figure 7-3. PRESCRIBED MELTING PERIOD



RSMAX = Residual moisture peak contribution
MLTBE = Day on which melting period begins
MLTEN = Day on which melting period ends

Figure 7-4. RATE OF MELTING

opportunity factor, OPINF, which represents the relative number and length of openings susceptible to infiltration. Pipe joints constitute the primary avenue for entry of infiltration. OPINF for an entire study area is determined by summing over all conduits the product of the perimeter of the conduit and the number of joints in each conduit, that is:

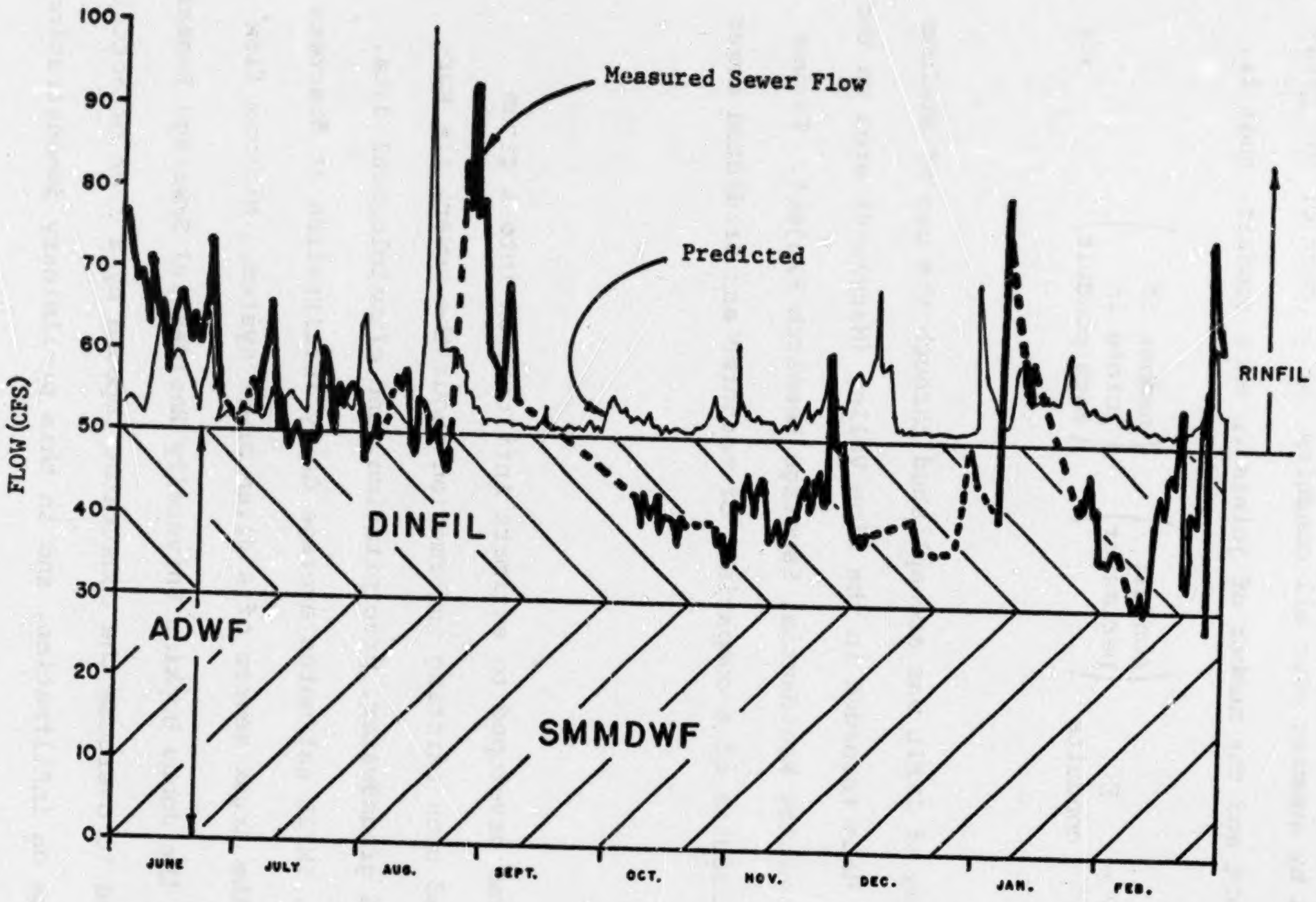
$$\text{OPINF} = \sum_{\text{conduits}} \begin{pmatrix}\text{conduit} \\ \text{perimeter}\end{pmatrix} \begin{pmatrix}\text{number of} \\ \text{joints in} \\ \text{each conduit}\end{pmatrix} \qquad (5)$$

## TEST APPLICATION

Preliminary testing of INFIL was accomplished through the use of minimum daily sewage flow data recorded in the Pine Valley (Maryland) area on the Johns Hopkins University Residential Sewerage Research Project. Figure 7-5 presents the results of a comparison of measured and predicted sewer flow.

## CONCLUSION

The INFIL model was developed to estimate infiltration into a given sewer system based upon existing information about the sewer, its surrounding soil and groundwater, precipitation, and climatological data. Using these data, INFIL estimates average daily infiltration at discrete locations along the trunk sewers of a given sewer system. Minimum flow data supplied by the Johns Hopkins University Residential Sewerage Research Project were used to overcome the constraint imposed by a lack of actual flow measurements on infiltration, and in this preliminary demonstration, the model proved to be satisfactory.



RINFIL = Wet weather infiltration
DINFIL = Dry weather infiltration
ADWF = Average daily dry weather flow
SMMDWF = Accounted for sewage flow

Figure 7-5. TEST RESULTS, PINE VALLEY (MARYLAND)

The INFIL model is not intended to represent direct or illegal connections to a combined or separate sewer system. The short response time of such connections is better represented through adjustments in the impurviousness ratio of the Runoff Model.

# SECTION 8

## TRANSPORT MODEL

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## SECTION 8

## TRANSPORT MODEL

### OBJECTIVES

Simulation of urban rainfall-runoff-quality processes necessarily involves consideration of flow in storm and/or combined sewers. Adequate representation of the response of urban catchments to precipitation inputs thus requires a proper representation of unsteady, non-uniform, free-surface flow which is predominant throughout sewers. In addition, prediction of flow quantities at various points within sewer systems, rather than only at the outfall, is desirable in order to study the effect of possible surcharging, overflow and diversion structures, proposed system modifications, and, in general, the overall capacity of sewer systems to handle storm runoff. Such a routing model may then serve also as a prerequisite to studies of quality parameter behavior in sewer systems.

### Sewer Routing

An accurate representation of velocity and depth of flow (or flow rate and flow area) along a sewer system can be found from the solution of the "shallow water" or "St. Venant" equations that are the general governing equations for gradually varied, unsteady flow (Ref. 1):

$$\frac{\partial y}{\partial x} + \frac{v}{g}\frac{\partial v}{\partial x} + \frac{1}{g}\frac{\partial v}{\partial t} = S_o - S_f \qquad (1)$$

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad (2)$$

where $y$ = Depth

$v$ = Velocity

$x$ = Longitudinal distance

$t$ = Time

$g$ = Gravitational acceleration

$S_o$ = Invert slope

$S_f$ = Friction slope

$Q$ = Flow rate

$A$ = Flow area

Eq. 1 is the dynamic equation and Eq. 2 is the continuity equation, with the assumption of no lateral inflows. In the absence of abrupt transitions (e.g., hydraulic jumps, bores), it is commonly accepted that solutions to the St. Venant equations offer a good description of the actual flow phenomena (Refs. 1,2). Unfortunately, solutions must be obtained numerically, and are usually arrived at using the method of characteristics (Refs. 1,2,3). This technique has been applied to sewer systems (Ref. 4) and has been found to be too consumptive of computer time for general applications. Thus, approximate techniques must be used in order to obtain a satisfactory flow routing method at a reasonable expenditure of computation time.

Traditional computations of flows and depths in sewer systems involve the assumption of uniform flow and normal depth in the computation of extremes and means and do not attempt to follow the progress of individual storm hydrographs in the system (Ref. 5). Eagleson (Ref. 6) has shown the applicability of the unit hydrograph concept to the prediction of hydrographs at the outfall of sewered areas. Owing to the nature of unit hydrograph techniques, such a method produces predicted hydrographs only at points in the system where prior flow measurements have been made and only for storms of specified duration. The technique also does not separate flow routing in sewer systems from flow routing of rainfall to sewer inlets. Unit hydrograph techniques thus lack the desired generality for application to all quantity and quality processes in sewer systems.

A recent approach to routing of urban runoff has been developed by Britain's Road Research Laboratory (Ref. 7). The RRL model, as it is called, has been applied to the prediction of outfall hydrographs with encouraging results (Ref. 8). However, it lacks the versatility required to model all components of a large system. In addition, tests conducted in the present research have shown it to lack accuracy when applied to long lengths of conduits with no intermediate inflows.

## Component Tasks

A routing method has thus been developed to compute hydrographs accurately at all points in a given sewer system while avoiding the necessity for solving the St. Venant equations. Evaluation of the model was subsequently done to judge its ability to reproduce measured hydrographs in real systems under a wide variety of situations with minimal time and expense to the user.

Component tasks performed by the Transport Model are to:

1. Coordinate user input data

2. Prepare routing sequence through trunk sewers
3. Coordinate surface runoff, infiltration, and wastewater quality and quantity inputs
4. Route flows through various conduit cross-sections and sewer appurtenances
5. Represent in-system storage, surcharging, and backwater conditions
6. Coordinate output to other programs and the user.

## THE MODEL SUBROUTINE

The computer program developed to perform the computations for flow routing in the sewers consists of the following 12 major subroutines: TRANS, TSTRDT, SLOP, FIRST, INFIL, FILTH, DWLOAD, INITAL, ROUTE, QUAL, PRINT, and TSTCST.

Subroutine TRANS provides the necessary liaison with the master Storm Water Water Management Model. TRANS also performs certain functions in relation to quantity routing which is described in the "User's Manual."

Subroutine TSTRDT, when used, reads in data to identify internal storage basins (those occurring at locations other than at the outlet of the system).

SLOP is called by TRANS to sequence the sewer element data for the later computations.

FIRST is called by TRANS to perform the initial computations of constants and flow parameters for each sewer element.

INFIL is called by TRANS to estimate and allocate sewer infiltration. INFIL is discussed in Section 7 of this report.

FILTH is called by TRANS to compute the average DWF quality and quantity. The DWF quantity and quality computations are discussed further in Sections 6 and 12, respectively, of this report.

DWLOAD is called by TRANS to compute the initial conditions of sedimentation within the sewer based upon the number of dry weather days prior to the storm.

INITAL is called by TRANS to initialize flows, areas, and pollutant concentrations to values corresponding to DWF plus infiltration.

ROUTE is called by TRANS to route the flow through the sewer elements.

QUAL is called by TRANS to route the pollutants through the sewer elements.

PRINT is called by TRANS to print the total hydrographs and pollutographs for the desired elements.

TSTORG is called by ROUTE to compute flow storage within the sewer system. TSTORG is discussed further in Section 9.

## THEORETICAL DEVELOPMENT

### Overall Model

*Input Data.* To categorize sewer systems conveniently prior to flow routing, each component of a system is classified as a certain type of sewer "element." All elements in combination form a conceptual representation of a system in a manner similar to that of links and nodes.

Elements may be classified as conduits, manholes, lift stations, storage units, or other appurtenances by specifying element types, e.g., manhole (type 16), circular conduit (type 1), and storage unit (type 19). Conduits themselves may be of different types depending upon their geometrical cross-section (e.g., circular, rectangular, horseshoe, etc.). Data input has been structured so as to allow element description on a "one element per data card" basis.

During preliminary development of the Transport Model, it was noted that the number of elements used to represent any given sewer system would necessarily have to be limited to less than 150 to conserve computer core space; approximately half of the elements would necessarily be devoted to non-conduits, leaving only 75 to conduits. To overcome this limitation, it was decided to defer consideration of the numerous and smaller lateral conduits and to aggregate remaining trunk conduits, if necessary. So as not to neglect the effect of lateral sewers upon flow routing, it was agreed that they would be considered as special gutters and, therefore, incorporated into surface runoff routing. Any necessary aggregation of remaining trunk conduits will be left essentially to judgment on the part of the user. However, an upper limit of approximately 3,000 feet will avoid inaccuracy in flow routing due to extremely large conduit length. Conduit elements of less than this length would be defined whenever significant changes in cross-section, slope, roughness, or incoming branches occur at shorter distances. Although criteria ultimately relate to changes in conduit flow-area relationships, it remains for the user to determine which changes are significant enough to warrant defining a new conduit element. Reference to Figure 8-1 should assist in describing sewer elements by showing an example sewer element schematic.

Non-conduit elements serve as convenient boundaries in the finite difference scheme used in conduit flow routing by providing for flow input, diversion, and storage. The most useful type of non-conduits are manholes, providing input locations for surface runoff, wastewater, infiltration, and upstream conduit flows and storage of temporary surcharging from downstream conduits. Thus, manholes would logically be placed as the most upstream element on each branch of the trunk system to allow flow input. All conduits should be linked by non-conduits, either to represent real manholes in the system, or to delineate conveniently changes in slope, roughness, etc., of adjoining conduit sections.

Element Sequencing. An extremely flexible sequencing routine was developed to order elements for routing. As a result, sewer elements defined on a sewer system schematic, such as shown in Figure 8-1, may be numbered in any order desired. By requiring that each element data card include that element's number and the numbers of elements directly connected upstream, elements can then be ordered within the computer by progressively searching for elements for which all upstream elements have been sequenced for routing. This progressive searching sequences elements regardless of the order in which element data are input. Thus, flexibility has been introduced by allowing both random numbering and random ordering of inputting elements.

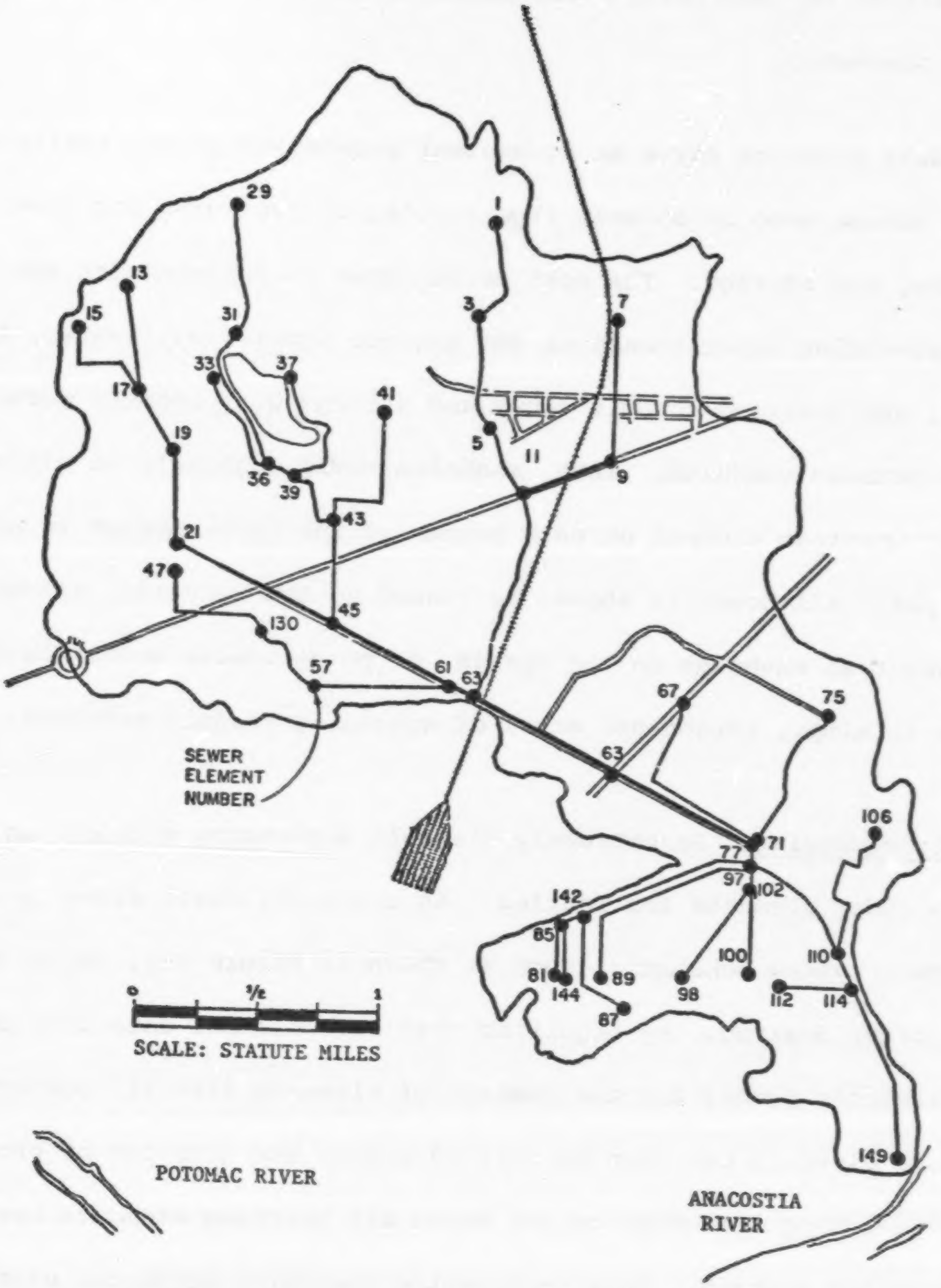

29
1
13
15
3
7
31
33
37
41
17
19
5
11
9
36
39
43
21
47
45
130
57
61
63
67
75
63
106
71
77
97
102
142
85
100
110
81
144
89
98
112
114
87
149
SEWER
ELEMENT
NUMBER
0
½
1
SCALE: STATUTE MILES
POTOMAC RIVER
ANACOSTIA
RIVER

Figure 8-1. SEWER SCHEMATIC FOR THE KINGMAN LAKE (WASHINGTON, D.C.) STUDY AREA

Inflows. Non-conduit element numbers nearest to the desired input point for wastewater and surface runoff are used to coordinate inflow. Wastewater flows estimated for subareas within the study area are assumed to enter at non-conduits located near the center of each subarea. Surface runoff estimated for each subarea within the study area are assumed to enter at non-conduits located at the downstream point of each subarea. For convenience, infiltration flows estimated for each conduit are allowed to enter at the upstream end of each respective conduit.

## Flow Routing by the Transport Model

The Transport Model Routing Technique. The continuity relationship, Eq. 2, is put into finite difference form, with reference to Figure 8-2. The subscript j denotes upstream conditions of flow, Q, and area, A, and the subscript j + 1 denotes downstream conditions. The subscript n denotes conditions at the previous time-step, and the subscript n + 1 denotes conditions at the new time-step. The time derivative will be weighted $w_t$ at point j + 1 and $1 - w_t$ at point j. The spatial derivative is weighted $w_x$ at time-step n + 1 and $1 - w_x$ at time-step n. The continuity equation is then

$$\frac{(1 - w_t)(A_{j,n+1} - A_{j,n}) + w_t\,(A_{j+1,n+1} - A_{j+1,n})}{\Delta t} + \frac{(1 - w_x)(Q_{j+1,n} - Q_{j,n}) + w_x\,(Q_{j+1,n+1} - Q_{j,n+1})}{\Delta x} = 0 \qquad (3)$$

The equations will ultimately be solved by starting at the upstream end and working downstream. Thus, at each time-step, the only unknowns will be the flow and area at the downstream end of the conduit length. Eq. 3

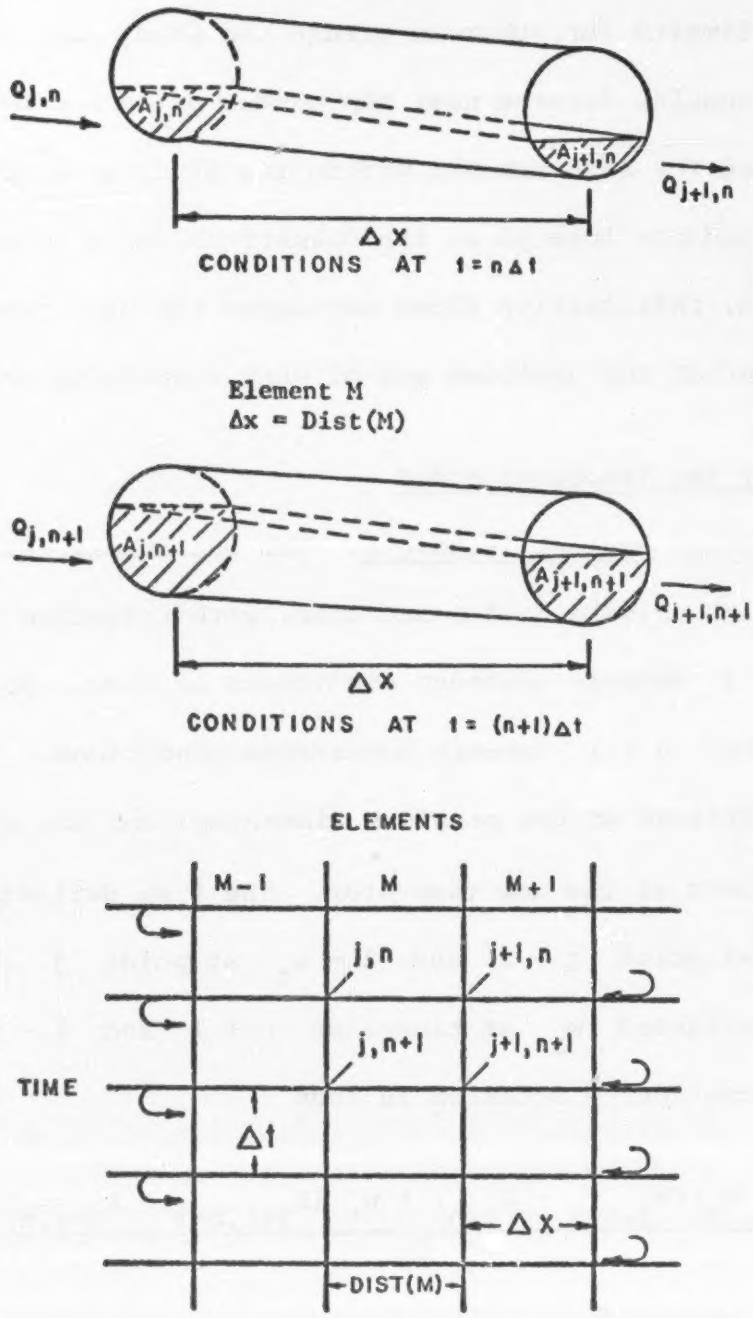

$Q_{j,n}$
$A_{j,n}$
$A_{j+1,n}$
$Q_{j+1,n}$
$\Delta X$
CONDITIONS AT $t = n\Delta t$
Element M
$\Delta x$ = Dist(M)
$Q_{j,n+1}$
$A_{j,n+1}$
$A_{j+1,n+1}$
$Q_{j+1,n+1}$
$\Delta X$
CONDITIONS AT $t = (n+1)\Delta t$
ELEMENTS
M-1
M
M+1
j,n
j+1,n
j,n+1
j+1,n+1
TIME
$\Delta t$
$\Delta X$
DIST(M)

Figure 8-2. FINITE DIFFERENCE DEFINITION SKETCH FOR ELEMENT M, ROUTING THROUGH ALL ELEMENTS AT EACH TIME-STEP

can be rearranged to isolate the downstream flow and area.

$$Q_{j+1,n+1} + \frac{\Delta x}{\Delta t}\,\frac{w_t}{w_x}\,A_{j+1,n+1} + \frac{\Delta x}{\Delta t\ w_x}\,[(1-w_t)(A_{j,n+1} - A_{j,n}) - w_t\,A_{j+1,n}] + \frac{1-w_x}{w_x}\,(Q_{j+1,n} - Q_{j,n}) - Q_{j,n+1} = 0 \qquad (4)$$

Eq. 4 is now normalized using values of flow and area for conditions of the conduit flowing full, denoted by the subscript f. The dimensionless area is denoted by $\alpha$,

$$\alpha \equiv \frac{A}{A_f} \qquad (5)$$

and the dimensionless flow rate by $\psi$,

$$\psi \equiv \frac{Q}{Q_f} \qquad (6)$$

Eq. 4 can then be written

$$\psi_{j+1,n+1} + C_1\alpha_{j+1,n+1} + C_2 = 0 \qquad (7)$$

where

$$C_1 \equiv \frac{\Delta x\ w_t\ A_f}{\Delta t\ w_x\ Q_f} \qquad (8)$$

and

$$C_2 \equiv \frac{\Delta x\ \ A_f}{\Delta t\ \ w_x\ Q_f}\,[(1-w_t)(\alpha_{j,n+1} - \alpha_{j,n}) - w_t\ \alpha_{j+1,n}] + \frac{1-w_x}{w_x}\,(\psi_{j+1,n} - \psi_{j,n}) - \psi_{j,n+1} \qquad (9)$$

Since the parameters $C_1$ and $C_2$ will be known at each time-step, Eq. 7 can be solved if a second relationship between the flow rate and area is known. Such a relationship can be found from Eq. 1 after first defining the friction slope, $S_f$, in the usual manner, using Manning's equation.

$$S_f \doteq \frac{V^2}{\frac{1.49^2}{n^2} R^{4/3}} = \frac{Q^2}{\frac{1.49^2}{n^2} A^2 R^{4/3}} \tag{10}$$

The variable R denotes the hydraulic radius, and n is Manning's roughness. The simplest result available from Eq. 1 is then obtained when all the terms on the left hand side are neglected. The resulting uniform flow relationship is

$$Q = \frac{1.49}{n} AR^{2/3} S_o^{1/2} \tag{11}$$

If Eq. 11 is normalized by the full-flow values of Q, A, and R, there results

$$\psi \equiv \frac{Q}{Q_f} = \frac{A R^{2/3}}{A_f R_f^{2/3}} = f(\alpha) \tag{12}$$

The normalized flow rate will be a function only of the normalized area since the hydraulic radius is a known function of the flow area for a given geometry. The relationship of Eq. 12 is illustrated in Figure 8-3 for three conduit shapes; it may be similarly derived for any conduit cross-sectional geometry.

In the development of the Transport Model, the first technique tried was simply to use Eq. 11 (leading to Eq. 12), as the required second

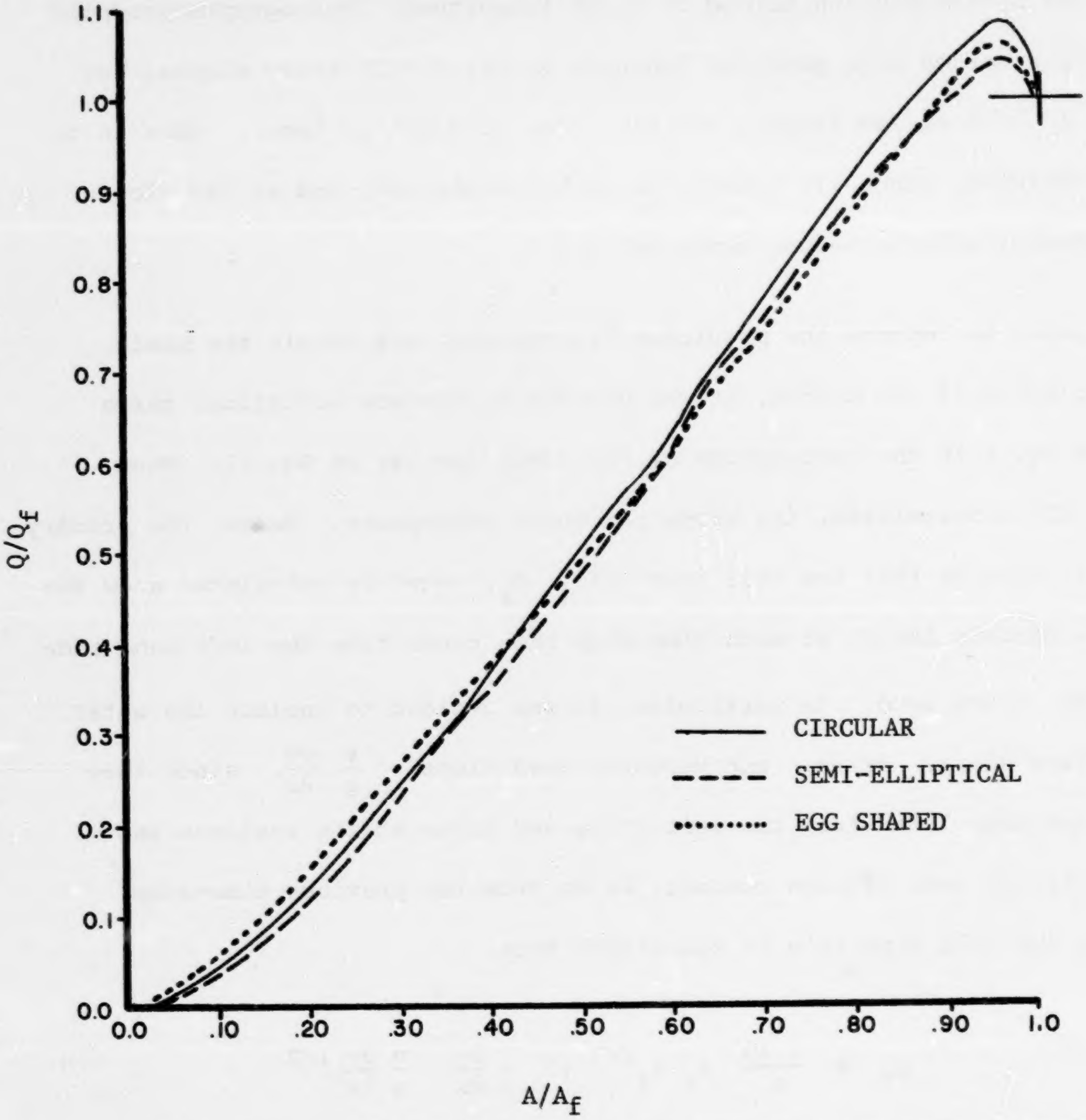


Figure 8-3. NORMALIZED FLOW-AREA RELATIONSHIP FOR UNIFORM FLOW

relationship between flow and area. When Eq. 12 is substituted into Eq. 7, the resulting non-linear algebraic equation is readily solvable by the Newton-Raphson method or other techniques. Hydrographs produced by this method were good for conduits on relatively steep slopes, but not as good at low slopes, (on the order of 0.001 or less). This is to be expected, since all dynamic terms are neglected, and at low slopes, backwater effects become appreciable.

In order to improve the predicted hydrographs, yet retain the basic simplicity of the method, it was decided to include additional terms from Eq. 1 in the calculation of the slope for use in Eq. 11. When Eq. 11 is normalized, the slope parameter disappears. Hence, the primary difference is that the full flow rate, $Q_f$, must be calculated anew for each conduit length at each time-step when terms from the left hand side of Eq. 1 are used. In particular, it was decided to include the water surface slope, $\partial y/\partial x$, and velocity head slope, $\frac{v}{g}\frac{\partial v}{\partial x}$, since they can be calculated from the velocities and areas at the upstream and downstream ends of each conduit, known from the previous time-step. Then the full flow rate is calculated thus:

$$Q_f = \frac{1.49}{n} A_f R_f^{2/3} \left(S_o - \frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x}\right)^{1/2} \qquad (13)$$

The derivatives are evaluated in finite difference form at time-step n.

$$-\frac{\partial y}{\partial x} - \frac{v}{g}\frac{\partial v}{\partial x} = -\frac{\partial}{\partial x}\left(y + \frac{v^2}{2g}\right) = \frac{y_{j,n} - y_{j+1,n}}{\Delta x} + \frac{v^2_{j,n} - v^2_{j+1,n}}{2g\Delta x} \qquad (14)$$

When this method was used, agreement between predicted hydrogrphs on low slopes and a standard of comparison (to be described later) was considerably improved. However, undesirable oscillations appeared in the predicted results. In an effort to eliminate the oscillations, an iterative procedure was developed in which, at each time-step, $Q_f$ was calculated two or more times for each conduit, using the values of velocity and depth calculated on the preceding iteration. The small remaining oscillations were removed by using a value of $Q_f$ taken as the average of the value from the previous iteration, and that just calculated. If the subscript $i$ represents the number of the iteration, then at iteration $i$,

$$Q_{f_i} = 1/2\, Q_{f_{i-1}} + 1/2\, \frac{1.49}{n} A_f R_f^{2/3} \left(S_o + \frac{y_{j,i-1} - y_{j+1,i-1}}{\Delta x} + \frac{V^2_{j,i-1} - V^2_{j+1,i-1}}{2g\, \Delta x}\right)^{1/2} \tag{15}$$

Four iterations were found to produce satisfactory results. When conduits have steep slopes, no iterations are required, and the invert slope, $S_o$, can be used alone. In order to avoid the possibility of a square root of a negative number, the sum of the slope terms is not allowed to be less than $S_o$. Since the only change is in the calculation of $Q_f$, it is still necessary only to substitute Eq. 12 into Eq. 7 and solve for $\alpha$ and $\psi$. The values of $A$ and $Q$ are subsequently obtained from Eqs. 5 and 6.

Stability of the Finite Difference Scheme. The stability of the method can be investigated using the techniques of Ref. 9, when the equations are linearized. It is found that the scheme is unconditionally stable (i.e., for any choice of $\Delta x$ and $\Delta t$ ), for $w_t$ and $w_x$ both greater than 0.5, neutrally stable for $w_t$ and $w_x$ equal to 0.5, and unstable (i.e., errors grow with time) for $w_t$ and $w_x$ less than 0.5. After numerous trials, a value of 0.55 was chosen for $w_t$ and $w_x$ since it resulted in the best attentuation of the hydrograph peaks, as well as insured stability.

Surcharging and Flow Routing in Manholes. Flow routing is accomplished in manholes by specifying that the outflow equals the sum of the inflows. Head losses at manholes are not considered a factor in trunk sewer flow routing, but could be accounted for simply by increasing the roughness coefficient of adjacent conduits. Unless diverted to an internal (in-system) storage element, surcharge volume from a downstream conduit is stored at the upstream manhole. Surcharged flows offer an additional flow input whenever excess flow capacity develops in the downstream conduit.

Flow Routing at Lift Stations. Lift stations are assumed to operate when wet well volume reaches a designated value which actuates a single, constant-discharge pump. Pumping continues until the wet well is emptied. Flooding caused when wet well capacity is exceeded is not considered.

Use of Flow Dividers. Flow dividers are programmed to perform in either of two ways. The first and simplest type assumes that diverted flow equals zero until an overflow setting is exceeded, after which all flow in excess of the setting is diverted. Junctions for relief sewers, bypasses, and other overflow structures occasionally perform in this manner.

Flow may also be diverted over a weir, in which case overflow is proportional to head above the weir taken to the 1.5 power. To determine head above the weir, flow depth through divider elements is assumed proportional to the flow rate. This head is calculated by multiplying the distance between the weir height and the top of the structure with the ratio of the incoming flow to the maximum possible flow. Side weirs as well as end weirs may be modeled by inputting the appropriate weir constant as data.

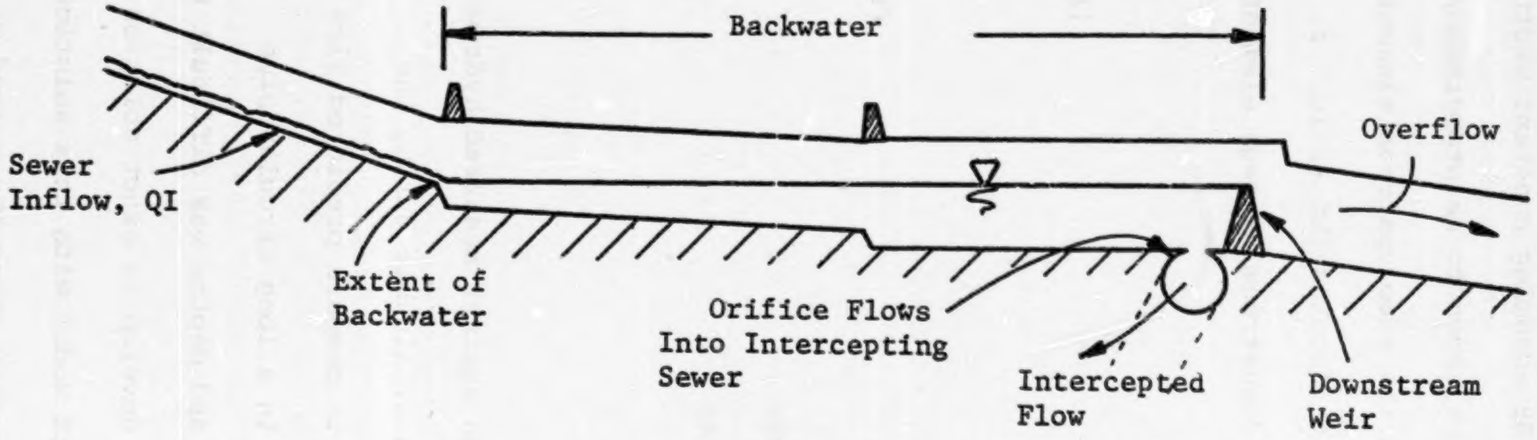
Flow Routing at Internal Storage Units. Flow through storage units located elsewhere than at the downstream point of a sewer system is routed by specifying a type 19 element. Storage units may be natural or man-made with weir and/or orifice outlets. Three routing options exist, namely, routing flow without quality, routing flow and quality assuming plug flow, and routing flow quality assuming complete mixing during each time-step.

The internal version of the Storage Model differs in structure from external storage (see Section 9) so as to allow more than one storage unit to serve on a time-step basis as a transport element. Also, as storage units at upstream locations will probably be smaller, no quality improvements, such as by settling, are modeled within internal storage. Refer to the Storage Model discussion (Section 9) for further description of storage routing.

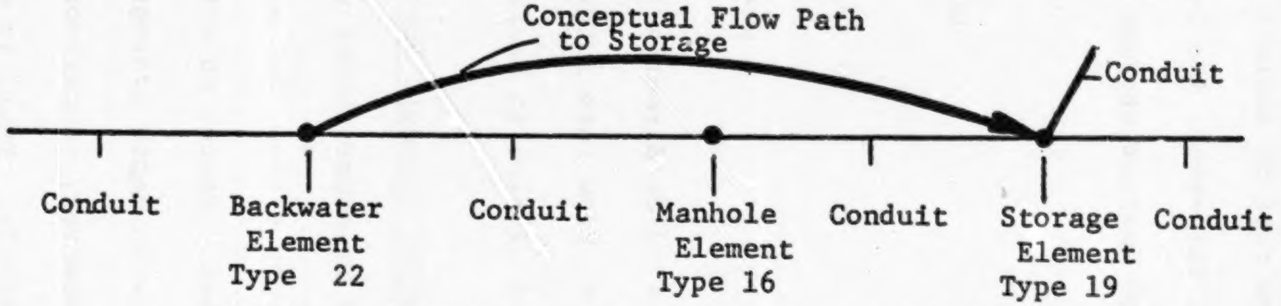
Modeling of Backwater Conditions. An accurate simulation of transient backwater conditions is not possible without an exact solution of the St. Venant equations simultaneously throughout the sewer system. The Transport Model does, in fact, include backwater effects for flow routing in a given conduit length by means of the iterative scheme described previously. However, since upstream conduits are routed independently of those downstream and no transfer of water depths is made, a continuous backwater profile is not simulated. The results (shown in Figures 8-5, 8-6, and 8-8) have shown the Transport Model to be very adequate in spite of this limitation.

However, backwater effects due to ponding as a result of a flow control structure can be simulated by a combination of conduit routing and in-system storage. The primary effect of such ponding is for upstream flows to be "felt" at the flow control structure earlier than if they were to flow normally through the intermediate conduits. The Transport Model simulates this effect by first assigning an extent of backwater by projecting the water surface horizontally upstream from the flow control structure at maximum depth until intersecting the invert slope. It is recognized that the actual extent of backwater as determined from an exact calculation would most probably be further upstream than indicated above. However, the intention is to model the "reservoir" or "ponding" effect only, and the assumption of a horizontal water surface slope considerably simplifies this task. A "backwater element," which is actually a special type of flow divider, is then placed at the location of the extent of backwater, and a storage element is placed at the location of the flow control structure. The concept is illustrated in Figure 8-4.

The backwater element is programmed to pass flow directly to the storage element or to the intermediate conduits, depending upon the amount of flow currently stored in the backwater region. In this way the upstream


Backwater
Overflow
Sewer Inflow, QI
Extent of Backwater
Orifice Flows Into Intercepting Sewer
Intercepted Flow
Downstream Weir

SEWER PROFILE


Conceptual Flow Path to Storage
Conduit
Conduit
Backwater Element Type 22
Conduit
Manhole Element Type 16
Conduit
Storage Element Type 19
Conduit

SCHEMATIC REPRESENTATION OF STORAGE IN THE TRANSPORT MODEL

Figure 8-4. TYPICAL IMPLEMENTATION OF A BACKWATER ELEMENT

flows are "felt" at the flow control structure (as simulated in the operation of the storage element) earlier than if they were routed totally through the intermediate conduits. By assuming constant width for upstream conduits, backwater length can be shown to be proportional to the square root of current storage within the same upstream elements. Then at each time-step, the backwater element computes the ratio, r, of current to maximum storage volume in the downstream storage element. Then

$$QO1 = QI \sqrt{r} \tag{16}$$

and

$$QO2 = QI - QO1 \tag{17}$$

where QO1 = Flow directly into storage unit

QO2 = Flow into intermediate conduits

QI = Inflow to backwater element

## TEST APPLICATION (Conduit Routing Method)

Ideally, the Transport Model should be tested against measured hydrographs in a real sewer system, but such data were unattainable during the testing phases. Hence, an attempt was made to compare predicted flow hydrographs with depth hydrographs measured in a long circular pipe (Ref. 3). However, comparison of the flows and depths was difficult to do quantitatively. Thus, it was decided to develop an exact solution of Eqs. 1 and 2, and to compare the Transport Model with this solution. Consequently, a solution was attained numerically using the method of characteristics in a manner similar to that described in Refs. 4 and 10. This was tested against the measured depth hydrographs of Ref. 3 and found to be in very good agreement. The characteristics solution was subsequently used as a basis of comparison of the Transport Model.

Comparisons of hydrographs predicted using the Transport Model and the characteristics solution can be seen in Figures 8-5 and 8-6 for the different time-steps and distance increments shown in Figure 8-7. The results are good even for the long total length of conduit used in the tests. Use of a long test section with no intermediate inflows was chosen to simulate the most unfavorable simulation conditions since results would be expected to improve when there were numerous inflows of known hydrographs along the test section. A further comparison was made with the published analytical results of Ackers and Harrison (Ref. 11) for a 1-foot diameter pipe on a slope of 0.001. The results are equally good, as shown in Figure 8-8.

These tests indicate the Transport Model to be an accurate and reliable tool for flow routing in sewers.

A comparison of actual and predicted hydrographs of a real sewer system can be found in Volume II of this report. A further description of the solution technique and computer program can be found in the "User's Manual."

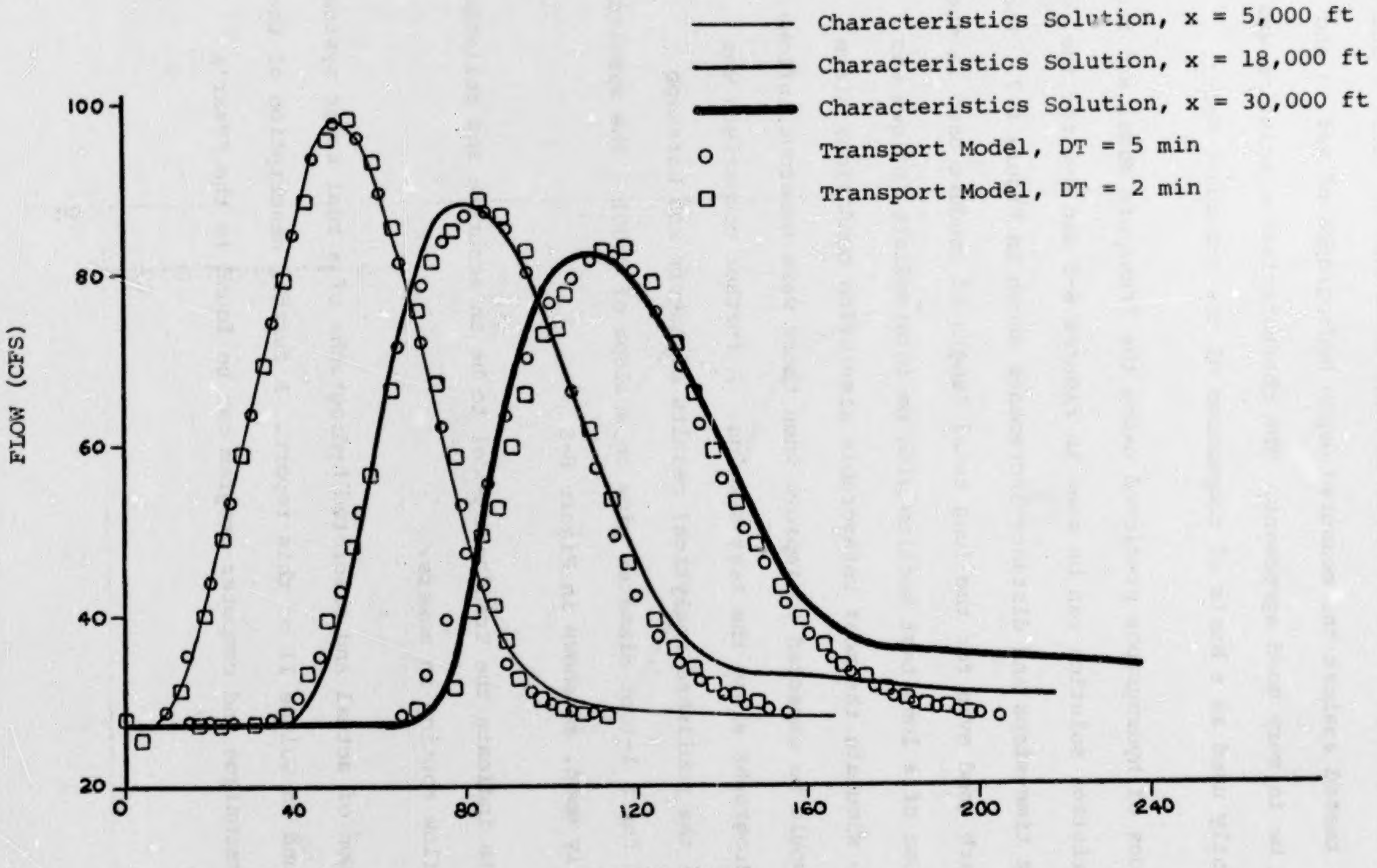


Figure 8-5. COMPARISON OF TRANSPORT MODEL AND EXACT SOLUTIONS FOR PIPELINE CONSISTING OF 8 CONDUIT LENGTHS

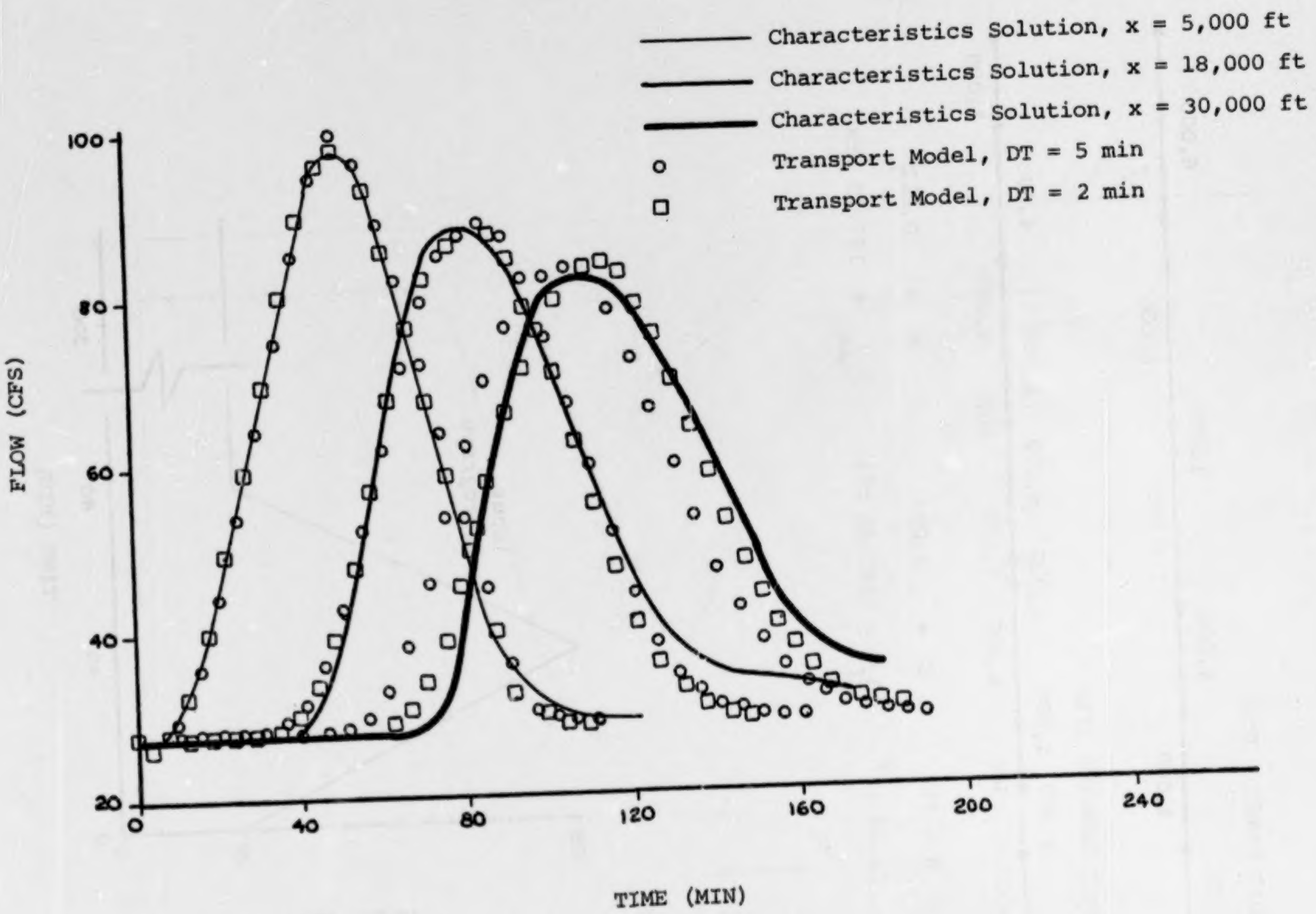
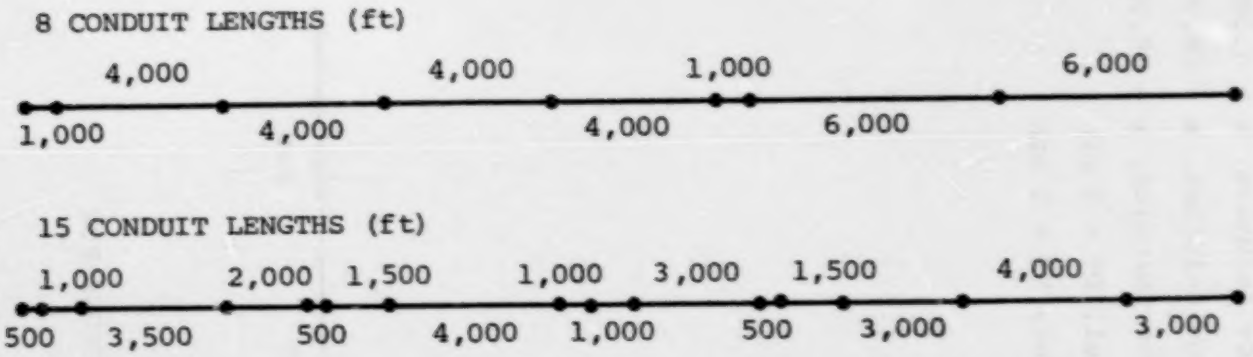


Figure 8-6. COMPARISON OF TRANSPORT MODEL AND EXACT SOLUTIONS FOR PIPELINE CONSISTING OF 15 CONDUIT LENGTHS


8 CONDUIT LENGTHS (ft)
4,000
4,000
1,000
6,000
1,000
4,000
4,000
6,000
15 CONDUIT LENGTHS (ft)
1,000
2,000
1,500
1,000
3,000
1,500
4,000
500
3,500
500
4,000
1,000
500
3,000
3,000

DIAM = 6.0 ft S = 0.001 N = 0.012

$A_f = 28.27 \text{ ft}^2$ $Q_f = 145.48 \text{ cfs}$ $Q_{max} = 157.12 \text{ cfs}$

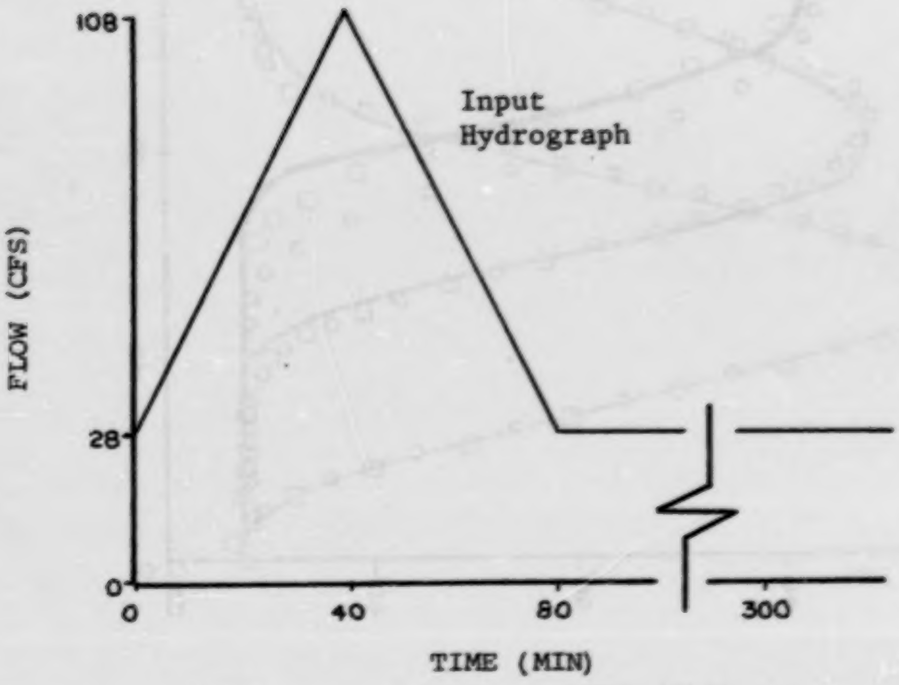


Figure 8-7. HYPOTHETICAL INPUT FOR ROUTING COMPARISONS

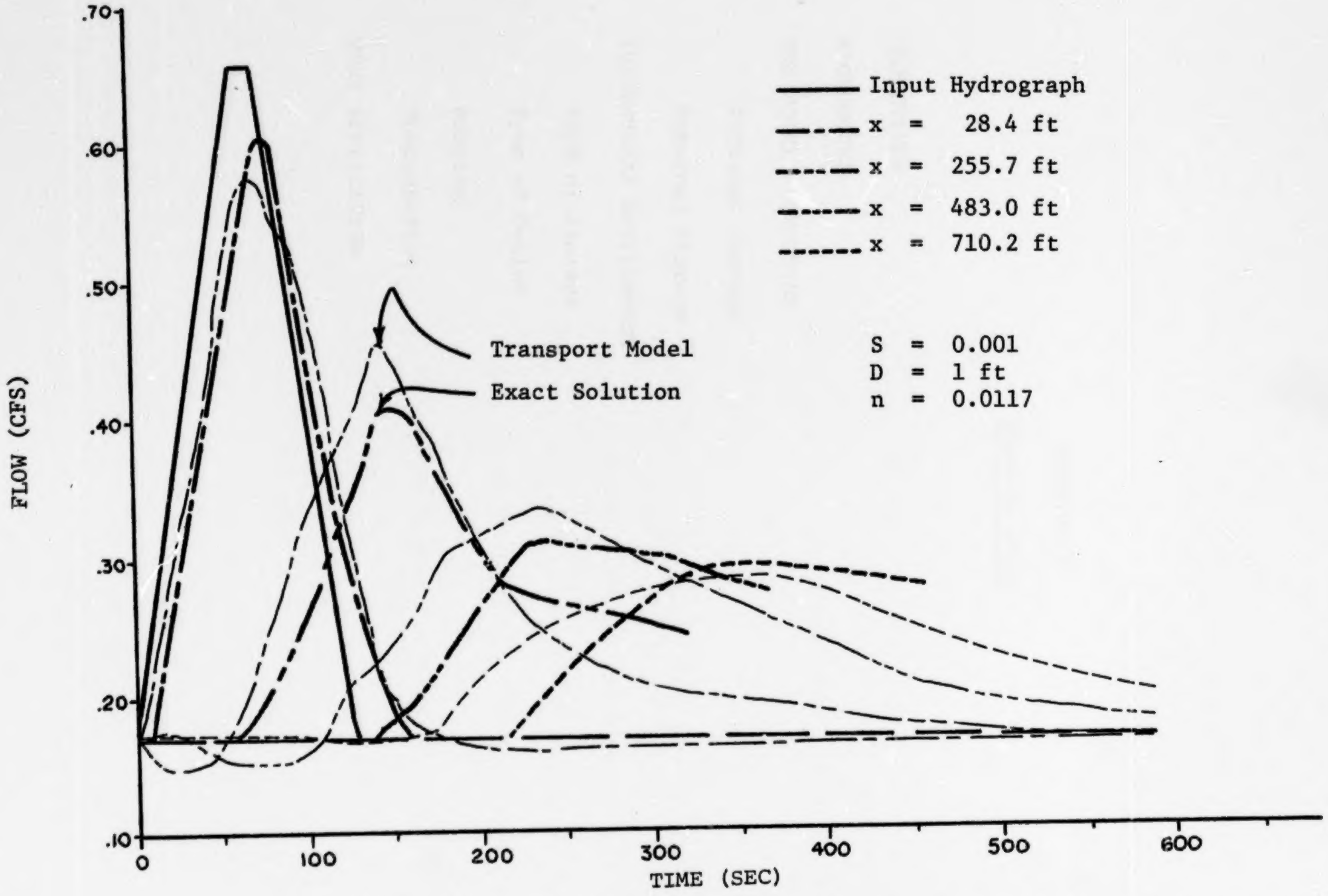


Figure 8-8. COMPARISON OF TRANSPORT MODEL WITH EXACT SOLUTION OF ACKERS AND HARRISON

# SECTION 9

## STORAGE MODEL

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## SECTION 9

## STORAGE MODEL

### OBJECTIVES

The objectives of the Storage Model are to:

1. Determine the modifying effects (attenuating or withholding) of various types (intentional or unintentional) of the storage element upon flows through sewer systems.
2. Simulate the movement of storm water within a storage unit.
3. Provide data required for the estimation of the costs of the specified storage installation.

### BACKGROUND

The functions and benefits of storage may be described as:

1. Reduction of the peak flow rate, thus
   - a. Reducing the size needed for a proposed storm drain.
   - b. Eliminating the need for a relief storm drain.
   - c. Enabling the handling of a more intense (lesser frequency) storm.
2. Reduction of pollutants in overflows by:
   - a. Capturing the solids contained in the first flush.
   - b. Providing continuous sedimentation and detention on a flow-through basis after storage volume has been filled.

These two uses of storage may not be compatible. For example, to reduce the peak it might be desirable to hold the storage in reserve until the design capacity of the conduit is almost reached and then to divert the excess flow to off-line storage. On the other hand, to trap the first flush of solids, the storage must be put in operation early, and it may be full by the time the peak flow arrives.

To capture the first flush in large drainage basins in flat country may require a considerable number of small storage tanks located on the principal branches upstream from the outlet. Just such a solution was proposed for large combined sewers in Buffalo by Riis-Carstensen in 1962 (Ref. 1).

In the past, most storage basins were designed as settling tanks and thus, in addition to storage, provided some degree of treatment through sedimentation. Although the design of new storage basins may not be optimal in terms of sedimentation theory, it is anticipated that some degree of treatment will be provided. Recognizing this limitation, the external version (see below) of the Storage Model assumes, however, that a storage basin will function efficiently as a settling tank. Treatment of pollutants by sedimentation is discussed in Section 15.

Two alternative versions of storage, requiring different modifications to a basic program, were considered for this study. The first, known as "internal" storage, was as one or more small storage tanks located on major upstream branches of the transport (sewer) system. Quality improvement (settling) within these smaller units was neglected; the units were modeled as elements of the transport system (see Section 8).

The second version, known as "external" storage, was as a major installation at the sewer system outfall. Many other methods of sewage treatment besides sedimentation might be provided in addition to this storage (see Section 15).

With either version, the first running of the Storage Model in a particular study project was visualized to include only existing storage reservoirs. After reviewing the results, and available locations for new storage reservoirs, the engineer would select the location and characteristics of proposed storage units. The data would then be read into the computer and the model re-run.

After reviewing these re-run results, additional runs could be made using reservoirs of different sizes and possibly adding or subtracting reservoirs at some locations until an acceptable solution is obtained.

The development of the Storage Model was based on the basic storage equation (outflow = inflow ± change in storage) with provisions for alternate facilities, such as holding/routing; artificial/natural; alternate inlet and outlet controls (weir, orifice, pumping); and alternate locations of installation. While the theory required to simply model the above features is quite fundamental and well understood, the nature of the flow through the storage unit may be a far more involved process. Types of through-flow considered include: plug flow, complete mixing, and short-circuiting.

## THE MODEL SUBROUTINE

The Storage Model processes inflow hydrograph and pollutograph data on a step-by-step basis. These data would usually have been generated by the Transport Model; it would also be possible to separately process data based on observed values. The time-step method of execution was adopted to simplify incorporation of the program into the Transport Model (internal storage).

Options of fixed (pumped) or variable (weir or orifice) outflows are provided, together with alternative types of flow through the reservoir, such as plug flow and complete mixing.

### Internal Storage

With the internal version of storage, one or two small storage tanks may be located on major upstream branches of the transport system. Quality improvements, such as sedimentation in the tanks, cannot be modeled with this version. The characteristics of the unit(s) to be modeled are specified via the Transport Model for internal storage.

The simulation proceeds on a time-step basis within the Transport Model, thus enabling the flows in all transport elements, including storage units, to be solved at each time-step, as the inflows become known.

If the water depth in any storage element exceeds at any time a previously specified maximum value, the unit is deemed to have flooded and the modeling is discontinued.

## External Storage

With the external version of storage, only a single, and usually major, installation may be modeled at the sewer system outfall.

The characteristics of the unit are specified, and the execution of the simulation is governed, by the Storage Block (see "User's Manual") in this case. Within the Storage Block many other methods of sewage treatment may be specified (see Section 15) in addition to the sedimentation that must occur within the storage unit.

The model accepts as input the output hydrograph and pollutographs from the Transport Model which have been stored on a disk or tape. The Storage Block is driven by the Executive Model quite independently of other models (blocks); flow through storage and associated treatment proceeds on a time-step basis.

If the water depth in the storage unit at any time reaches a previously specified maximum value, an inlet control structure is assumed to bypass any additional flows that would otherwise further increase the depth.

Output from the Storage Block (hydrograph and pollutographs) is again stored on a disk or tape, for possible later use as input to the Receiving Water Model (both quantity and quality).

## THEORETICAL DEVELOPMENT

### Type of Storage

Two basically different types of storage are included. The first is that of an irregular (natural) reservoir. In this case 11 pairs of depth versus surface area data must be provided by the user. The second type of storage is that of a regular (artificial, man-made) unit. Here the user must provide information on the base area, circumference, and side slopes, from which the depth/surface area relation may be computed.

From the depth increments and areas, the depth versus storage relation is easily obtained in either of the above cases.

Type of Outlet

Orifice and weir outlets are included for gravity (variable rate) outflows, both separately and in combination.

For an orifice with known effective area $C_dA$ (coefficient of discharge times orifice area), the outflow (cfs) is computed from:

$$Q_{out} = C_dA\sqrt{64.4\ D} \qquad (1)$$

where D = Water depth (ft) above the orifice centerline

For a weir of known height (above the storage = 0 level) and length L (ft), the outflow (cfs) is computed from:

$$Q_{out} = 3.33\ LH^{1.5} \qquad (2)$$

where H = Head (ft) over the weir crest

Fixed-rate outflows are also included, under the pumped outflow option. In this case, the buffer volume in storage between pump start and stop depths is computed and compared with the volume capable of being pumped out per time-step. If the buffer volume is too small, a message is printed recommending remedial measures and warning that subsequent computations are possibly unreliable.

Routing

With gravity outflows, the program requires that the basic routing equation is satisfied, i.e., that

$$0.5(I_1 + I_2)Dt = 0.5(O_1 + O_2)Dt + (S_2 - S_1) \qquad (3)$$

where I = Inflow rate (cfs)

O = Outflow rate (cfs)

S = Stored volume (cf)

Dt = Hydrograph time-step size (sec)

and where subscript

1 represents conditions existing at the start of Dt

2 represents conditions existing at the end of Dt

Rearranging to place the unknowns at time 2 on the left hand side of the equation,

$$(0.5)O_2Dt + S_2 = 0.5\ (I_1 + I_2)Dt - ((0.5)O_1Dt - S_1) \qquad (4)$$

The quantity on the left hand side, given the FORTRAN name ATERM in the program, may thus be evaluated for a new time-step. Knowing the storage and outlet characteristics of the reservoir (discussed above), the relation between ATERM and $(0.5)O_2Dt$ may be obtained. This relation is stored in the computer in the form of 11 pairs of routing parameters; intermediate points are obtainable by simple linear interpolation.

From this selection the value of $(0.5)O_2Dt$ corresponding to the known ATERM is obtained; the required values of $S_2$ and $O_2$ for this reservoir and time are then easily found from these known quantities.

The hydrograph is routed through the reservoir by repeating this procedure for each time-step.

The initial conditions within the reservoir must be specified. For outlet controls such as the weir, with the initial storage below the weir crest, the program will simulate the filling period before outflow begins.

While routing the flows, the program accumulates the total volume of storage inflow and outflow during the storm. These volumes are afterwards compared with the initial and final volumes in storage, to provide a continuity check.

Through-Flow

Having solved for the outflow rates from the storage reservoir or tank, the nature of the flow inside the unit is of concern insofar as it governs the movements, detention times, and thus the releases of the accompanying pollutants.

The types of through-flow considered were: complete mixing, plug flow, and short-circuiting. Which, if any, of these would actually occur is a complex question, probably depending upon the shape of the storage unit, arrangement of the inlet and outlet(s), the flow rate, the pollutants load, and other factors.

With the option of complete mixing, the inflow is thoroughly mixed with the existing contents in storage, resulting in "averaged" qualities the outflow.

With the plug flow option, the inflow during each time-step, here called a plug, is labeled and queued through the storage volume. Transfer of pollutants between plugs is not permitted, but contributions of several plus are averaged in the outflow time-step. The program identifies the inflow plugs, or fractions thereof, which comprise each outflow plug; thus the pollutant load and detention time for each outflow fraction is easily found.

Short-circuiting could occur to considerably varied extents, depending even more upon the above-mentioned factors of shape and load, etc. For these reasons, the simulation of short-circuiting has not been included in this model.

Neither sedimentation nor scour are simulated within the Storage Model. The pollutant loads are only traced and redistributed as indicated above. With the external version of the storage program, sedimentation in storage is computed by the Treatment Model (Section 15).

## TEST APPLICATION

The Selby Street outfall in San Francisco, chosen to test the use of this model, serves a tributary area of some 3,800 acres. An interceptor normally carries dry weather flow to a treatment plant. The combined sewer outfall was built as three rectangular compartments, as shown in Figure 9-1. During storms these compartments may fill up and overflow a 100-foot long weir into San Francisco Bay (Ref. 2).

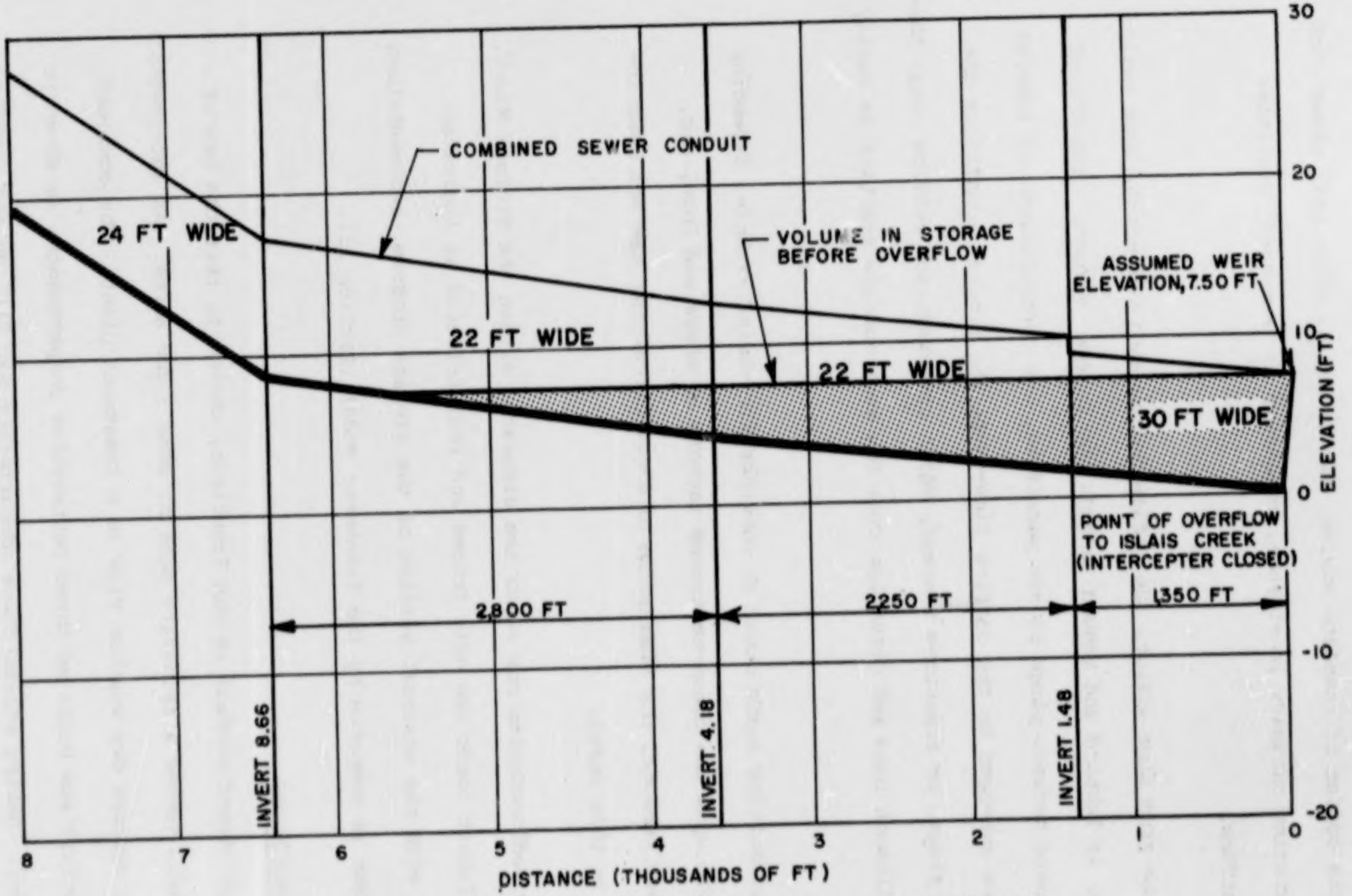


Figure 9-1. OUTFALL STORAGE, SELBY STREET, SAN FRANCISCO

The Storage Model as developed was used to calculate the effects of this external outfall storage on the combined outflow hydrograph.

Because of the more complex shape of this storage unit, the irregular type of storage was specified. The required data pairs of depth versus surface area were taken from Figure 9-1 and are presented in Table 9-1. More data points were taken above the weir crest (at 7.50-feet depth), where more sensitivity is preferred.

Table 9-1. STORAGE MODEL INPUT DATA, SEWER STORAGE BASIN, SELBY STREET, SAN FRANCISCO

| Data Pair Number | Depth, ft | Surface Area, sq ft |
|---|---|---|
| 1 | 0.00 | 0 |
| 2 | 2.00 | 49,900 |
| 3 | 5.00 | 102,200 |
| 4 | 7.50 | 136,200 |
| 5 | 8.00 | 129,700 |
| 6 | 8.50 | 121,100 |
| 7 | 9.00 | 112,600 |
| 8 | 9.50 | 114,500 |
| 9 | 10.00 | 116,000 |
| 10 | 10.50 | 109,200 |
| 11 | 11.00 | 102,000 |

Due to the elongated shape of this storage unit, plug flow is probably more appropriate than complete mixing for the simulation of pollutant movements in this case.

The inflow hydrographs and pollutographs used were output from a simulation with the Transport Model of the storm of November 6, 1966

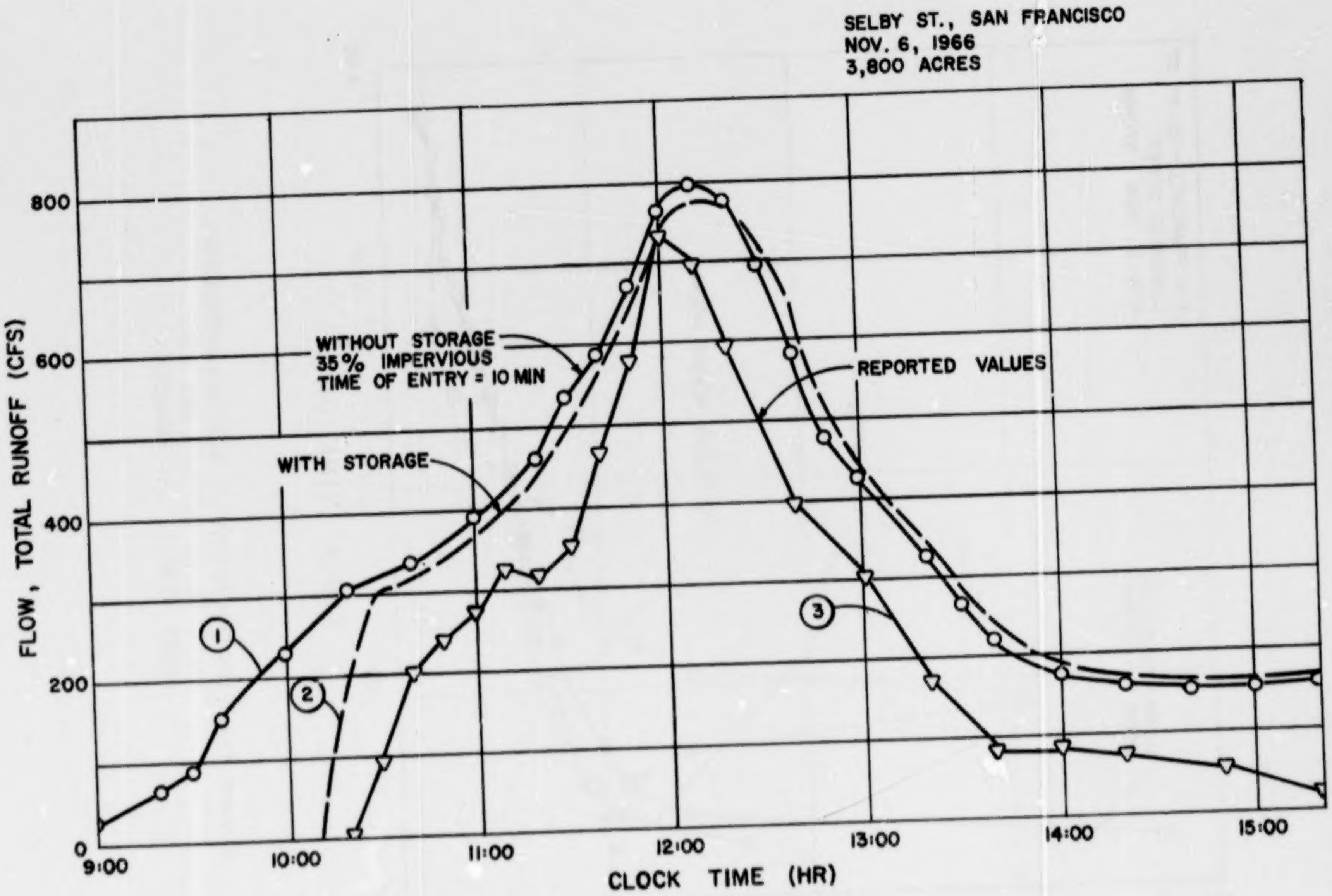
(Ref. 2) in the contributing sewer system. A 10-minute time-step was used.

The hydrologic results of the test application are presented in Figure 9-2. Hydrograph (1) is the input to the Storage Model, being the output from the Transport Model without storage. Hydrograph (2) is the output from the Storage Model, i.e., the result of routing hydrograph (1) through the sewer storage basin. Hydrograph (3) is the reported hydrograph (Ref. 2), which must include the storage effect.

Differences between hydrographs (2) and (3), the total volume of flow under the curves in particular, clearly result from assumptions made in the previous modeling and generation of hydrograph (1). However, the effects of storage as indicated by the differences between hydrographs (1) and (2) are as might be expected. In particular, the delay in the start of the rising limb while the reservoir was filling agrees well with the reported time that overflow started.

The continuity error was negligible, being less than one millionth of the total inflow.

The quality results of the test application are given in two further figures. Variations with time of BOD and suspended solids concentrations are presented in Figures 9-3 and 9-4 respectively. The observed concentrations (Ref. 2) are compared in both figures with the output from the Storage Model using both the complete mixing and plug flow options described earlier. Results are also given for a third through-flow model



Note: ①, ②, and ③ refer to hydrograph numbers.

Figure 9-2. HYDROGRAPH MODIFICATION PRODUCED BY OUTFALL STORAGE, SELBY STREET, SAN FRANCISCO

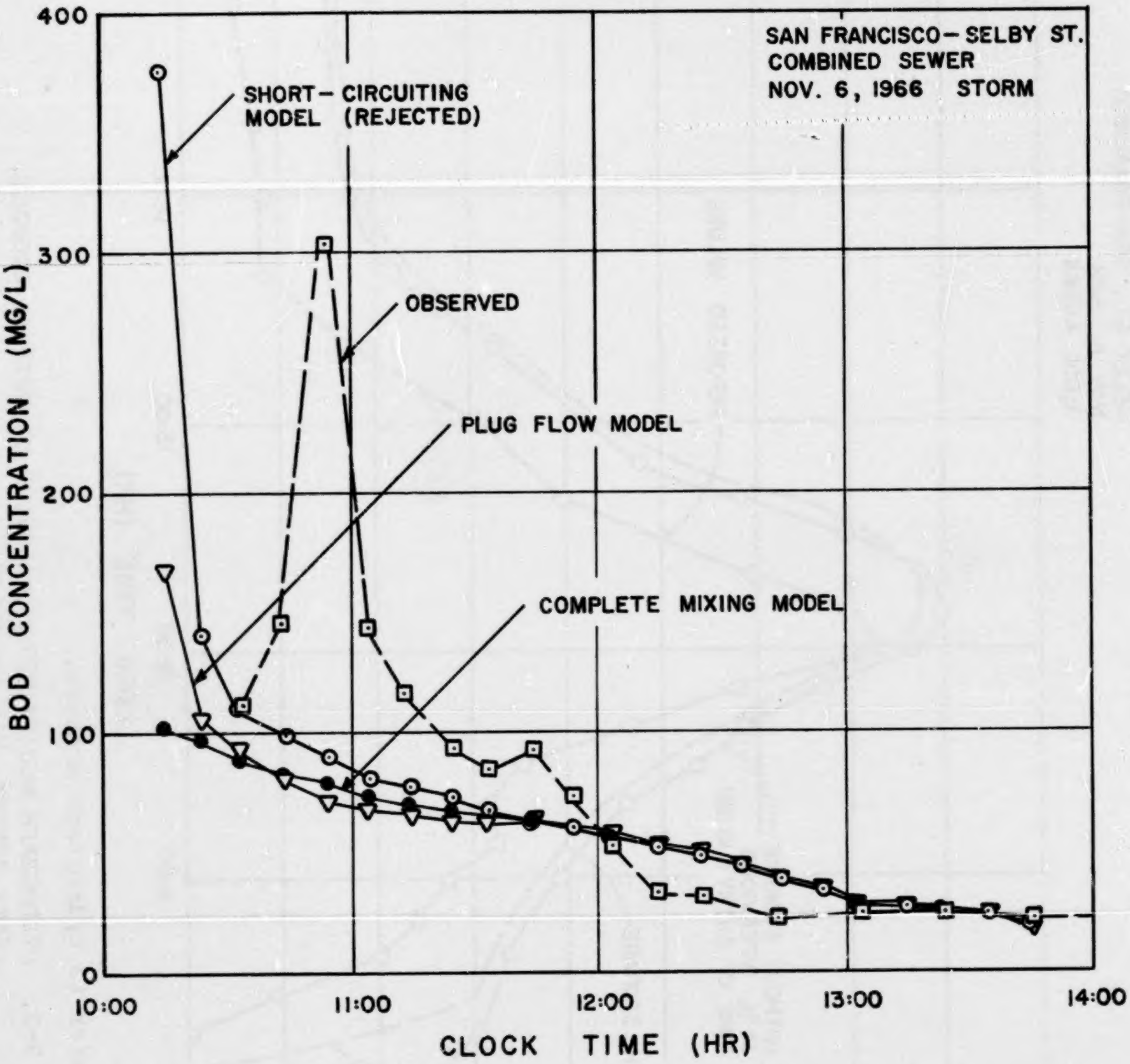


Figure 9-3. MODIFICATIONS TO BOD CONCENTRATION PRODUCED BY OUTFALL STORAGE, SELBY STREET, SAN FRANCISCO

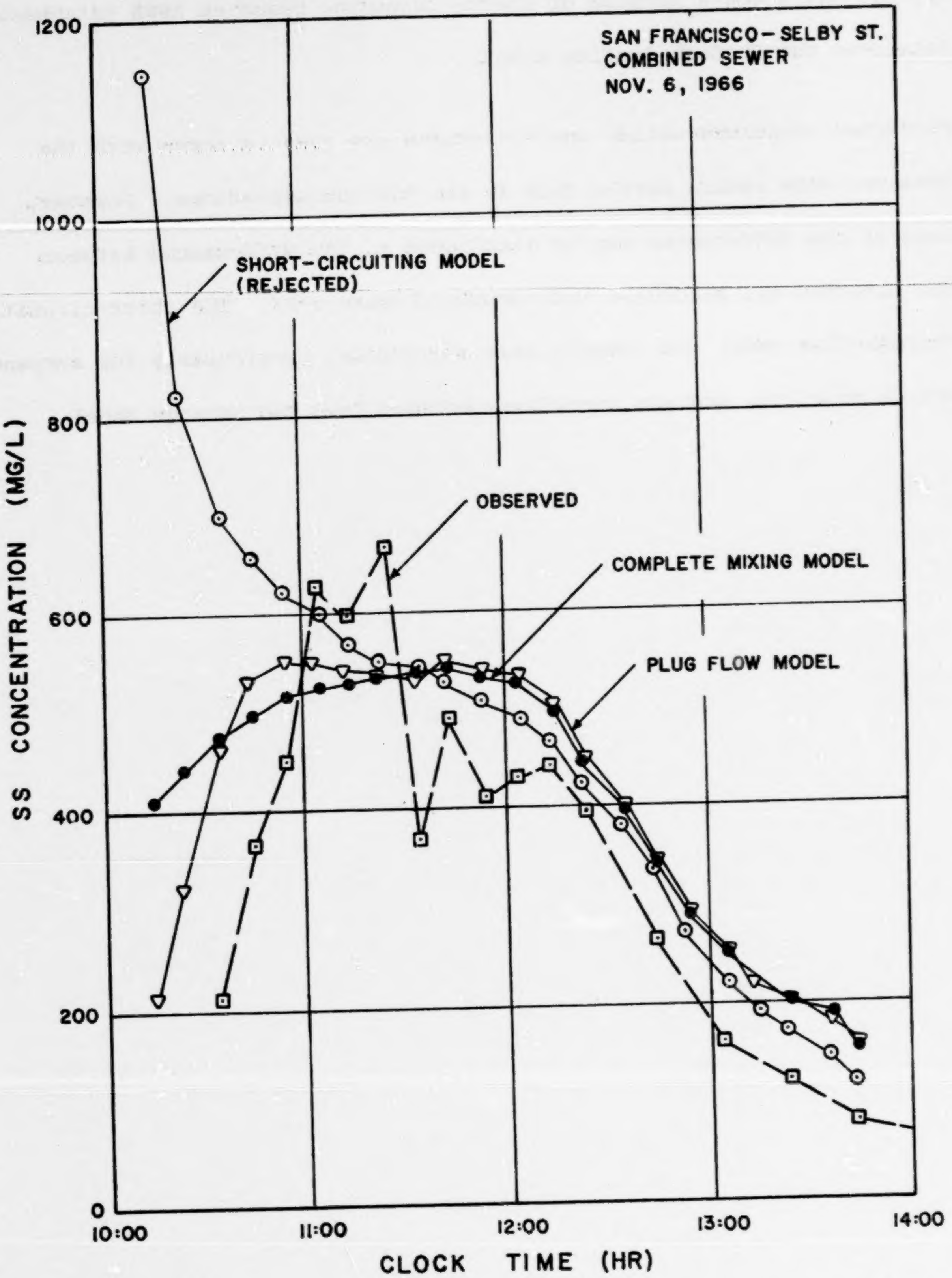


Figure 9-4. MODIFICATIONS TO SUSPENDED SOLIDS CONCENTRATION PRODUCED BY OUTFALL STORAGE, SELBY STREET, SAN FRANCISCO

option, which was a version of short-circuiting based on APWA catchbasin data (see the Surface Quality model).

Predicted suspended solids concentrations are seen to agree with the observed data rather better than do the BOD concentrations. However, some of the differences may be attributed to the differences between the observed and predicted hydrographs (Figure 9-2). The short-circuiting through-flow model was clearly less acceptable, particularly for suspended solids modeling, and was therefore excluded from the Storage Model.

# SECTION 10

## RECEIVING WATER QUANTITY MODEL

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# SECTION 10

## RECEIVING WATER QUANTITY MODEL

### OBJECTIVE

The objective of the Receiving Water Quantity model is to provide the hydraulic basis for the determination of the receiving water quality responses resulting from storm water pollution. The receiving water body may be an estuary, a stream, or a lake. The model simulates the movement of water for each type of water system.

The receiving water bodies may have different geometric configurations with various boundary conditions. To be of wide applicability, the model must use a system that can readily be adapted to any prototype condition by simply changing the input data.

For this purpose, the model represents the water body by a network of nodal points connected by channels. The nodal points and channels are idealized hydraulic elements which are characterized by parameters, such as surface area, cross-sectional area, length, and friction coefficient. Equations of motion and continuity can thus be applied to each element and solved simultaneously to produce a time-history of stage, velocity, and flow at the various points of the water system.

### RELATION TO STORM WATER PROGRAM

The receiving water body is the end point where the pollutional effects of storm water are to be evaluated. At this point, the runoff hydrograph at each outfall and associated pollutographs for various chemical constituents have already been computed by a sequential operation of the Runoff, Transport, and Storage Blocks of the Storm Water Management Model.

The nodal point of the receiving water system adjacent to an outfall will accept a runoff hydrograph input. The computed stage and flow conditions of the receiving water are then passed on to the Receiving Water Quality model for quality routing. The transfer of information between the programs is normally handled by interfacing storage devices such as magnetic tape written in compatible format.

## THE MODEL SUBROUTINE

Liaison between the Storm Water Management Model and the Receiving Water Quantity model is provided by subroutine RECEIV which accepts the hydrograph and pollutograph at the outfalls from the Transport Model. The model consists of six major subroutines, i.e., SWFLOW, INDATA, TIDCF, TRIAN, PRTOUT, and OUTPUT. Hydrograph information is used in the hydraulic computation and pollutograph information is passed on to the Receiving Water Quality model discussed in Section 14.

RECEIV calls SWFLOW which then takes over the control to perform hydrodynamic computations with the assistance of other subroutines. For example, INDATA is called to read input data pertaining to geometric information of channels and junctions and control parameters specifying the days of simulation, time interval of computation, and so forth. If the water body is a lake, coefficients for the weir formula at the downstream outlet will be read by SWFLOW. Otherwise, SWFLOW will call

TIDCF to determine the tidal coefficients describing the boundary constraint at the mouth of the estuary.

On the basis of the data received through INDATA, TRIAN uses the laws of trigonometry to interpolate geometric information for hydraulic elements of the water systems which otherwise would have to be deduced from a map manually.

Upon completion, SWFLOW calls PRTOUT to print hydraulic stage and flow behavior at selected points in the water system. It also calls OUTPUT to prepare computational results for graphical display by a printer plot routine. SWFLOW itself then writes detailed hydrodynamic information for subsequent use by the Receiving Water Quality model.

## THEORETICAL DEVELOPMENT

The hydrodynamic behavior of a water body influenced by external forces is governed by two fundamental equations, i.e., the equation of motion and the equation of continuity. A complete description of the movement of water requires a simultaneous solution of the two equations.

While the differential equations of motion and continuity have been described for various coordinate systems since the eighteenth century, the solutions have been developed only for limited cases with simple boundary conditions. For example, the flow problems and their analytical solution for the one-dimensional channel have been studied quite extensively for constant boundary conditions, such as a steady inflow and/or a downstream head control. When the inflow is variable, the problems become too complicated to be solved by conventional analytical methods. The degree of difficulty compounds further as the water body becomes so big that flow can occur in a two-dimensional plan or in a three-dimensional volume. Yet, these are the kinds of problems that need to be solved in the Storm Water Management Model.

Recently, these complex hydrodynamic problems have been approached by numerical methods assisted by advanced computer technology. One such approach, developed by WRE, has attempted to develop an iterative solution technique for the hydrodynamic equations to be performed by a computer.

While this hydrodynamic model has proven to be very versatile in dealing with various geometric and boundary conditions, it has never been used to accept a hydrograph input. The modification of the model to treat the transient hydrodynamic behavior and the associated problems caused by storm water inflow was considered the next logical step in its development.

## Geometric Representation

The computer model requires a nodal point and grid system for the representation of the prototype receiving water body. For a simple stream, one can conceptually represent the body of water by sectioning the river into reaches. A reach is defined as a channel where water flows from one end to the other. The end of the reach is further defined as a junction where two reaches meet.

If the head at the junction at any given time is known, the hydraulic gradient along the channel and thus the flow rate may be calculated.

By transferring water from one end to the other according to the computed flow, the head at the junction will subsequently be altered. Therefore, a stepwise hydraulic computation can be performed on a discretized system representing the water body.

To facilitate the hydraulic computation, it is necessary to supply the surface width, length, average depth, and friction coefficient for the channels and the surface area, head, and floor elevation for the junctions. The junction surface area is defined as one-half of the surface areas of the preceding and succeeding reaches; the floor elevation is defined as the depth of the bottom ground surface from a given datum plane such as mean low low water. All of these geometric parameters are readily measurable from a navigation map. It must be noted that the floor elevations are referenced to the same datum for all junctions.

For a lake or estuary, the water body may not be represented reasonably well by a single channel. In this case, the channel and junction system is used to form a grid covering the water body. Several channels can now meet at a junction, enabling the depiction of two-dimensional flow characteristics in a horizontal plan. Since the channel is the hydraulic element where water flows, the layout of the principal channel must conform with the direction of current as closely as possible. Figure 10-1 shows the geometric representation of a receiving water with some of the possible hydrologic inputs.

The shape of the grid system is flexible in that it can be orthogonal, triangular, or irregular. If the condition warrants, the use of a

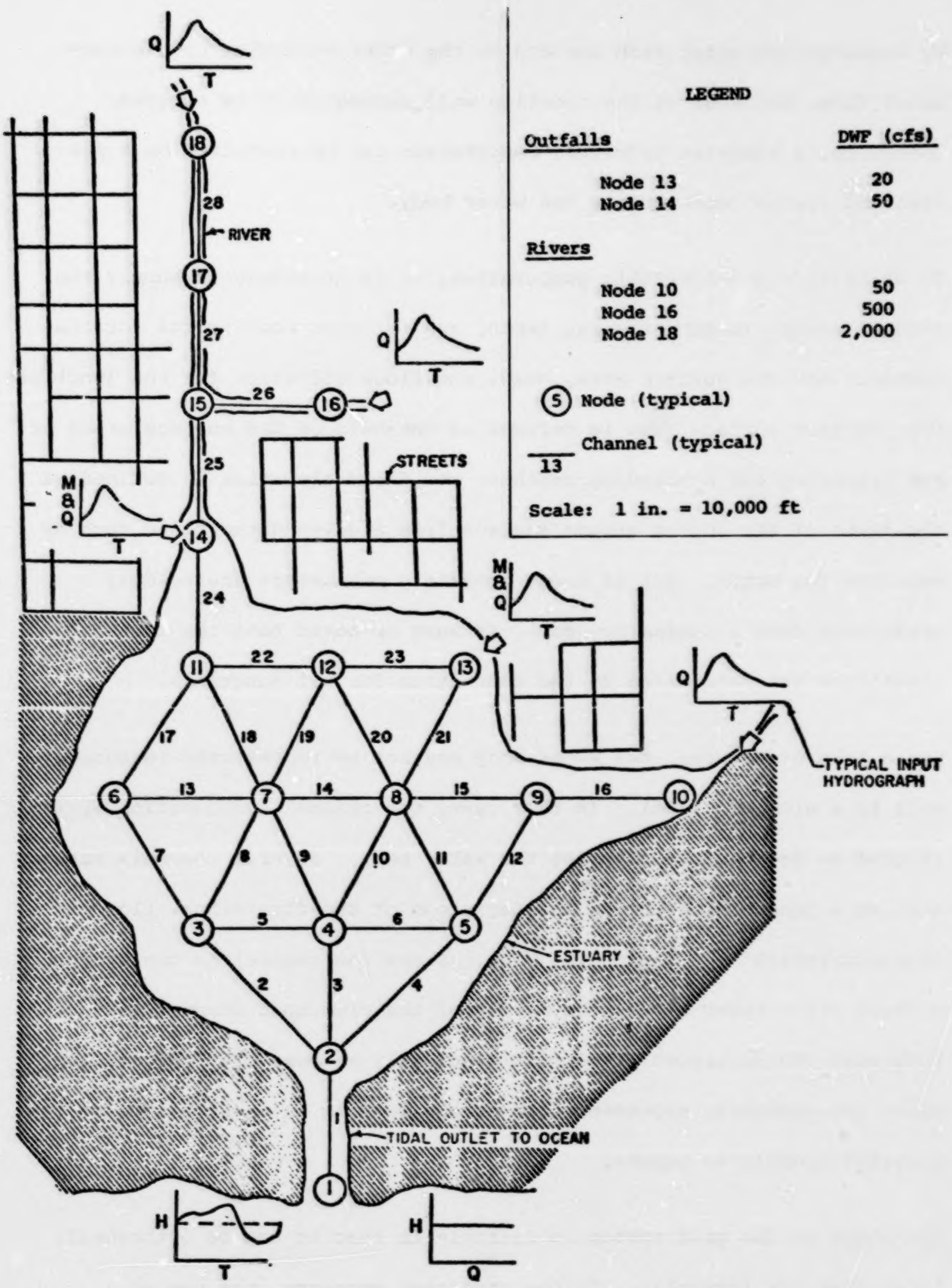
LEGEND
Outfalls
DWF (cfs)
Node 13 20
Node 14 50
Rivers
Node 10 50
Node 16 500
Node 18 2,000
5 Node (typical)
13 Channel (typical)
Scale: 1 in. = 10,000 ft
Q
T
RIVER
STREETS
M & Q
ESTUARY
TYPICAL INPUT HYDROGRAPH
TIDAL OUTLET TO OCEAN
H
T
H
Q
1
2
3
4
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Figure 10-1. GEOMETRIC REPRESENTATION OF A RECEIVING WATER

triangular shape is recommended. Many of the data for a triangular system can be deduced from the laws of trigonometry and thus the data preparation is greatly simplified.

The channel width in a grid system is not as well defined as that for a simple stream. The present working definition of the channel width is the distance between two lines parallel to the channel and passing through the centroids of two triangles sharing the channel. If it is not a triangular grid, the centroid of whatever shape is used will still be used for measuring the channel width.

The length of the channel is governed by a stability criterion of the numerical integration technique. This criterion was described by Garrison et al. as follows (Ref. 1):

$$\left( V + \sqrt{g \frac{A}{B}} \right) \frac{\Delta t}{\Delta x} \leq 1 - \frac{gn^2 \, |V|}{2.21 \, R^{4/3}} \, \Delta t \qquad (1)$$

where $V$ = Mean velocity

$A$ = Cross-sectional area

$B$ = Surface width

$g$ = Gravitational acceleration

$n$ = Manning's resistance coefficient

$R$ = Hydraulic radius

$\Delta t$ = Time interval

$\Delta x$ = Channel length

For channels, A/B is equal to depth h. The relationship can then be simplified to

$$\Delta t \leq \alpha \frac{\Delta x}{\sqrt{gh}} \tag{2}$$

where $\alpha$ = Proportionality constant

$\sqrt{gh}$ = Celerity of wave

Since the computational time interval is normally constant throughout the system, the deeper channel should have a longer length according to the expression. A channel length longer than necessary favors the stability, but it reduces the accuracy of hydraulic and quality computations. Experience has shown that good results can be obtained by an $\alpha$-value ranging from 0.7 to 0.8.

Solution Procedure

Basically, the hydraulic computations for each time interval proceeds as follows:

1. Compute the flow rate in each channel according to the hydraulic gradient and other hydraulic conditions existing at the beginning of a time interval.
2. Compute the rise or fall of the water surface (head) at each junction based on the channel flow and the importation or withdrawal of water at the junction.
3. Update the geometric and hydraulic conditions for the computation of the next time interval.

Thus, the iterative procedure uses the equation of motion to calculate the channel flow and then uses the equation of continuity to determine the junction head. Since the computations proceed step-by-step, the boundary conditions, such as the tide and the hydrologic input, can be varied from one time interval to the next.

Mathematically, the equation of motion for a one-dimensional channel can be written as follows:

$$\frac{\partial V}{\partial t} = -V\frac{\partial V}{\partial x} - g\frac{\partial H}{\partial x} - gS_f + gS_w \qquad (3)$$

where $V$ = Velocity

$t$ = Time

$x$ = Distance

$H$ = Water surface elevation measured from the datum plane

$g$ = Gravitational acceleration

$S_f$ = Energy gradient

$S_w$ = Wind stress

The energy gradient $S_f$ of turbulent flow is proportioned to the square of the mean velocity according to Manning's equation,

$$S_f = \frac{n^2 V |V|}{2.2 R^{4/3}} \qquad (4)$$

where $n$ = Friction coefficient

$R$ = Hydraulic radius of the channel

The wind stress can be approximated by the following expression:

$$S_w = \frac{K}{d} \cdot \frac{\rho a}{g\rho_w} U^2 \cos \psi \qquad (5)$$

where $K$ = Dimensionless coefficient with a value of 0.0026 (Refs. 2,3)

$d$ = Depth of flow

$\rho a$ = Air density

$\rho w$ = Water density

$U$ = Wind velocity

$\psi$ = Angle between the wind direction and the axis of the channel

The second equation necessary to complete the mathematical formulation of the problem is furnished by the continuity requirement at the junctions. This requirement states that the net effect of water flowing into the junction through channels or importation is to raise the water surface elevation at the junction, i.e.,

$$A_{sj} \frac{\partial H_j}{\partial t} = \sum_{i=1}^{k} Q_i + Q_j \qquad (6)$$

where $A_{sj}$ = Surface area associated with the junction j

$Q_i$ = Flow of a connecting channel

$Q_j$ = Water importation rate to the junction

The equation still applies for cases where the water surface elevation is lowered resulting from outflow.

Numerical solution of Eqs. 3 and 6 entails a rewriting of both equations in finite difference form. The initial value of various parameters at time $t_o$ is used to determine the rate changes of flow and water head during a short period of time (integration interval $\Delta t$). Based on the rate change, the next value is computed and the whole procedure is repeated.

Various numerical methods have been developed to solve this type of initial value problem. The present model uses a modified Runge-Kutta technique where the interval of integration is divided into four. The intermediate time-step computations improve the stability and accuracy of the model.

## Transient Hydrodynamics

As stated previously, the model requires initial values at time $t$ to compute values of the next time interval, $t + \Delta t$. With the detailed finite element representation of the receiving water body, however, there is difficulty in obtaining the starting conditions for each channel and junction.

This difficulty was resolved by starting the program from a standstill condition with the mean tide elevation at each junction as the initial head. The specifiable boundary conditions, such as the tide or head relationship at the governing downstream point, are then imposed on the model to proceed with the computation. In the first two days of computation, the system is in a transient state toward a dynamic steady state in which the stage and current throughout the system are synchronized to the given tidal condition.

The program, equipped with all necessary hydraulic information to start the actual simulation, will then accept the storm water inflow for the calculation of hydrodynamic responses in the receiving water. The system will again be in the transient state until the storm is ended and its residual effect diminished.

## Alternate Boundary Conditions

The distinction between estuaries and streams or lakes in the present model is strictly a problem of downstream boundary conditions. In estuaries, the boundary condition is usually a specified tide at the mouth, such as the Golden Gate of San Francisco Bay. A lake is constrained by a head-flow relationship, (i.e., weir equation) of the outlet. Rivers can be a special case of either estuaries or lakes depending again on the downstream condition, i.e., a constant head or a stage-flow relationship of the channel.

For the lake, thermal stratification may occur at certain times of the year, influencing the hydraulic response. Prediction of thermal behavior in conjunction with the storm water inflow is beyond the scope of this study. However, the location of a thermocline can be expected to be known for a particular simulation through field measurement. For storm water purposes, one can then neglect the presence of water below the thermocline in estimating depths for the hydraulic computation.

## TEST APPLICATION

Actual data were not available for testing the hydrodynamic response of the receiving water under the storm water effect. Present model testing is therefore based on the engineering evaluation of the reasonableness of the answers. The general method of procedure, however, has been proven valid for several prototype conditions accepting the constant inflow (Refs. 4,5,6).

Several tests were made using the geometric system of Figure 10-1. As shown, each channel was 10,000 feet long, with a depth ranging from 15 feet at channel 28 to 20 feet throughout the open water body.

Figure 10-2 shows the input hydrographs applied to the system. Not shown in the figure are the hydrograph inflows to junctions 13 and 10 which are identical in shape and time to those of junctions 14 and 16, respectively, except that the flow coordinate is ten times smaller. These input hydrographs are superimposed on the dry weather flow of 2,000 cfs at junction 18, 500 cfs at junction 16, 50 cfs at junctions 10 and 14, and 20 cfs at junction 13.

For the test of an estuary case, a tidal wave was imposed on junction 1. A typical dynamic response of flow in channel 24 is plotted in Figure 10-2. The transient effect of storm water inflow is noted to push the flow pattern toward outflow (seaward) direction. The case shown herein is a shallow estuary with a strong tide at the mouth. Tidal influence, therefore, dominates the system behavior and yet the transient hydrodynamic effect caused by storm water inflow is discernible after the fifth day of simulation.

This response appears reasonable as do others when the system is considered a lake and a stage-flow relationship is specified for junction 1. Verification runs for the Receiving Water Model were made at three demonstration sites and are described in Volume II of the final report.

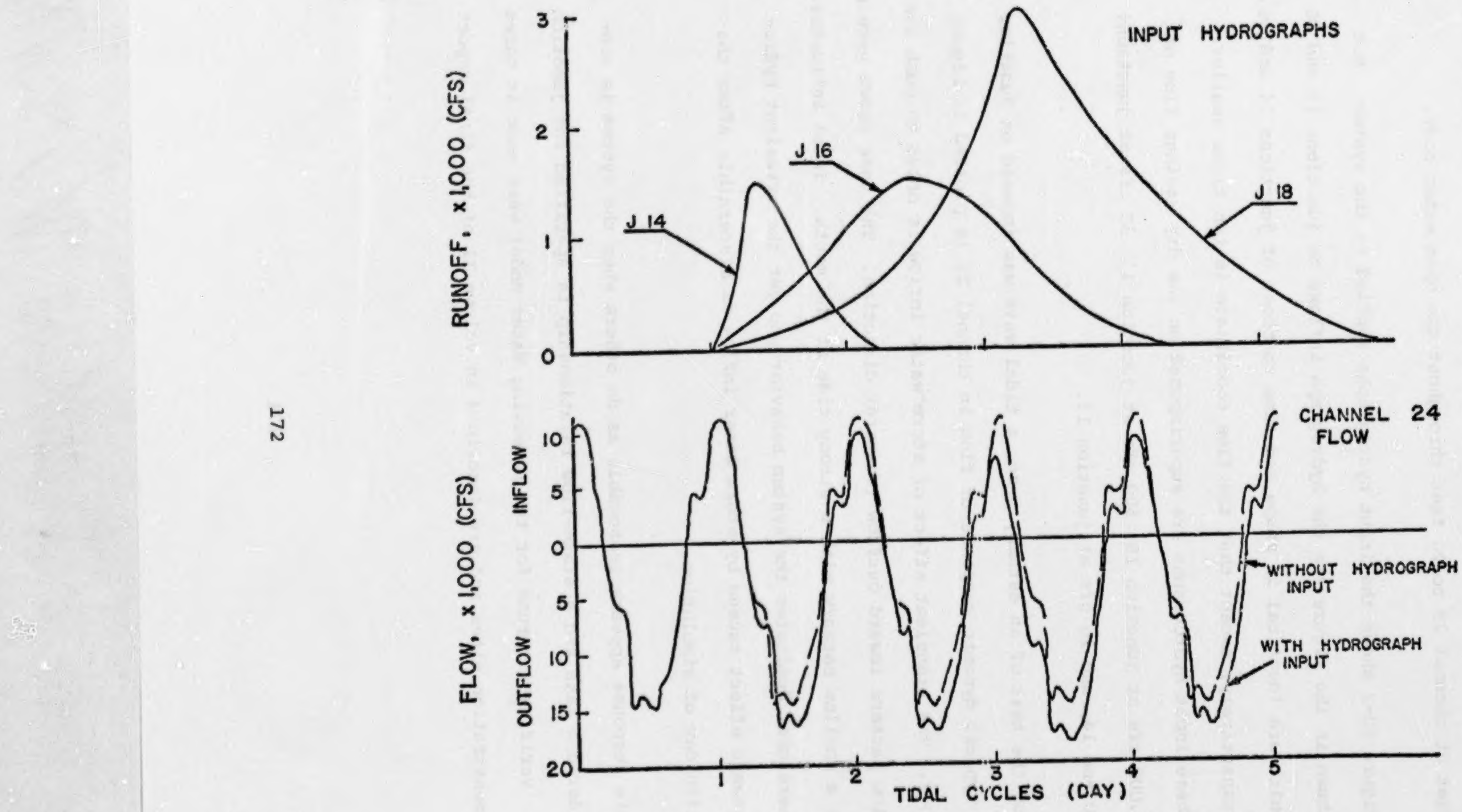

Figure 10-2. TEST SYSTEM FLOWS SHOWING HYDROGRAPH EFFECTS

# PART 3

# QUALITY SUBROUTINES

# SECTION 11

## SURFACE RUNOFF QUALITY MODEL

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# SECTION 11

## SURFACE RUNOFF QUALITY MODEL

### OBJECTIVES

The objectives of the Surface Runoff Quality model are to:

1. Identify the quality constituents or pollutants.
2. Determine the quantities of pollutants on each subcatchment prior to the storm event.
3. Determine the rate of removal of pollutants during a storm event.

### BACKGROUND

The literature contains many studies on the determination of the volume of surface runoff from both natural watersheds and urban sewered areas, and some notable compilations of the quality of overflows from combined sewers and the discharge of separate storm drains. These studies have amply demonstrated that storm water overflows contribute significantly to the overall water pollution problem and should be investigated. Only a very few studies, however, have included data on both runoff quantity and quality in sufficient detail so that the quality of direct surface runoff could be computed as a function of the runoff intensity. Consequently, in the development of the Surface Runoff Quality model it was necessary to rely on a theoretical approach, with the values of the coefficients determined by and checked against the available data. The development and preliminary verification of the theory is described in this section.

## IDENTIFICATION OF POLLUTANTS

Many different determinations are made in the analysis of wastewater to determine its pollution load. Pollutants may be classified as conservative or nonconservative. Examples of conservative pollutants are: total and volatile suspended solids, ammonia and organic nitrogen, phosphate, and oil and grease. Examples of nonconservative pollutants are BOD, COD, and coliform content. The modeling of the surface runoff quality from a drainage area assumes that all pollutants are conservative because of the short transport times involved. Pollutants may also be classified as soluble or nonsoluble. The modeling method will handle any pollutant for which data are available. To date, the data have permitted the modeling of BOD, suspended solids, and coliforms.

## THE MODEL SUBROUTINE

The model (subroutine SFQUAL) accepts as input the hydrograph of surface runoff developed for the subarea and produces three pollutographs, one of BOD and one of suspended solids, both in pounds of pollutants per time-step, and the other of coliforms in MPN per time-step. At the head end of the system, the hydrograph of surface runoff and the three pollutographs are delivered to the Transport Block which combines the hydrograph with the volumes of infiltration and dry weather flow, and the three pollutographs with the BOD, suspended solids, and coliforms contributed by the dry weather flow. The pollution contribution of the infiltration is assumed to be zero. The Transport Block routes the resultant hydrograph and three pollutographs down the sewer to the next inlet point, taking account of the volume of wastewater and pounds of pollutants--or in the case of coliforms, MPN--in the sewer at the start of the rain, as well as the changing volume of wastewater and the amount of pollutants stored in the sewer during each time-step. At the next inlet point, the contributions from the next subarea are added to the hydrograph and three pollutographs modified by passage through the Transport Block. Then, the combined hydrograph and three pollutographs are routed down the next stretch of sewer by the Transport Block. This process is continued at successive inlet points. A multi-branched sewer system is assumed.

The process is facilitated by initially expressing the pollutants in pounds or MPN per time-step. To obtain the concentration in mg/L or MPN/100 ml, it is only necessary to divide the total pounds in any time-step by the total volume of flow during the same time-step using the correct conversion factor. Both the pollutograph (which gives the pounds or MPN) and the hydrograph (which gives the volume of flow) must be for the same point in the sewer system. At the end of the subroutine SFQUAL, the pollutants are converted to lb/min or MPN/min for coliforms before being transferred to the Transport Block via output file.

## THEORETICAL DEVELOPMENT

### Initial Theory

At the start of the rain, the amount of a particular pollutant on surfaces which produce runoff (both impervious and pervious) will be $P_o$ , pounds per subarea. Assuming that the pounds of pollutant washed off in any time interval, dt , are proportional to the pounds remaining on the ground, $P$ , the first order differential equation is:

$$\frac{-dP}{dt} = kP \qquad (1)$$

which integrates to

$$P_o - P = P_o(1-e^{-kt}) \qquad (2)$$

in which $P_o - P$ equals the pounds washed away in the time, $t$ . In order to determine $k$ , it was assumed that $k$ would vary in direct proportion to the rate of runoff, $r$ , or $k = br$ . To determine $b$ it was assumed that a uniform runoff of 1/2 inch per hour would wash away 90 percent of the pollutant in one hour. This leads to the equation:

$$P_o - P = P_o(1-e^{-4.6rt}) \qquad (3)$$

where $r$ = Runoff rate (in./hr)

$t$ = Time interval (hr)

(Other assumptions for determining $b$ could be made but this original assumption has proven satisfactory in all test applications to date for urban areas.)

In using this equation, a uniform time-step $t$ is selected; values of $r$ are determined from the inlet hydrograph; and the equation is applied successively, the value of $P$ determined at the end of the $n$th interval becoming the value of $P_o$ at the beginning of the $(n+1)$th interval. The value of $r = 0.5(r_n+r_{n+1})$. (4)

The concentration in each interval may be determined by dividing $P_o - P$ by the runoff in cfs and multiplying by a constant which depends on the time interval. This constant equals 268 divided by the time interval in minutes. Since $r$ in inches per hour = cfs per acre, the runoff in cfs = $r$ times area in acres. The concentration may be expressed in the form of an equation as follows:

$$\text{Conc. in mg/L} = \frac{268}{\text{time-step in min}} \times \frac{(P_o - P)\text{ in lb}}{r\text{ in in./hr} \times \text{area in acres}} \qquad (5)$$

Note: in./hr times acres is equal to runoff in cfs.

Catchbasins as a source of pollution are considered later in this section.

## The Cincinnati Study

A paper (Ref. 1) describing the results of a study by the U.S. Public Health Service (USPHS) of a 27-acre area in Cincinnati served by separate sewers contains data which were used as a preliminary check on the foregoing theory. The preliminary trial runs indicated that the theory would produce figures of the right order of magnitude for a storm similar to the first 4 hours of the reported storm of March 16, 1963, if appropriate values of $P_o$ were assumed. Further, the concentration of suspended solids and BOD were shown, as in Table IV of the paper, to decrease with the length of the storm, although not as rapidly. Since Table IV gives mean values which would not apply precisely to a particular storm, the agreement with the general trend was considered a satisfactory check. The comparative figures are not reproduced herein because refinements in the estimation of the values of $P_o$ and in the application of the method to suspended solids make the original figures of historical interest only. The Cincinnati paper proved invaluable in the further development of the method as described hereinafter.

## Estimation of Pollutants on the Ground, $P_o$

The Cincinnati study gives estimates of the pounds of pollutants in the storm water runoff "based on essentially complete measurement of rainfall and consequent runoff and quality at the study site during September through November 1962, and March through September 1963, projected to average annual rainfall at Cincinnati" as shown in Table 11-1.

Table 11-1. ESTIMATED ANNUAL RUNOFF OF POLLUTANTS FROM CINCINNATI AREA

| Constituent | Lb/acre/yr |
|---|---|
| SS | 730 |
| VSS | 160 |
| COD | 240 |
| BOD | 33 |
| $PO_4$ | 2.5 |
| Total N | 8.9 |

Source: S. R. Weibel et al., "Urban Land Runoff as a Factor in Stream Pollution," July 1964 (Ref. 1, excerpt from Table V).

Dividing the figures in Table 11-1 by 365 provides average figures for the proportion of the daily accumulation that appears in the surface runoff. These may be considered reasonable averages for urban conditions in the Midwest and East, i.e., for urban areas having similar annual weather and rainfall patterns. They do not, however, provide the amount of pollutants on the ground at the start of a rain as affected by antecedent conditions, street cleaning practices, and neighborhood characteristics.

These necessary data were found in the American Public Works Association (APWA) study (Ref. 2) and are incorporated in the model. The study gives the daily accumulation of dust and dirt, D/D, per 100 feet of curb and its soluble BOD content for various types of neighborhoods in Chicago. These are reproduced in Table 11-2. In this section and in the model terminology the terms curb and gutter are used interchangeably.

Table 11-2. DAILY DUST AND DIRT ACCUMULATION IN CHICAGO AREA

| Land Use | Dust and Dirt, D/D, lb/day/100 ft of curb | BOD of D/D, mg/gram |
|---|---|---|
| Single family | 0.7 | 5.0 |
| Multiple family | 2.3 | 3.6 |
| Commercial | 3.3 | 7.7 |
| Industrial | 4.6 | 3.0 |
| Average of above, weighted | 1.5 | 5.0 |

Source: APWA, "Water Pollution Aspects of Urban Runoff," January 1969, WP-20-15 (Ref. 2, p. 56).

The figures in Table 11-2 represent the accumulation on the streets and in the gutters of the dust and dirt fraction of the litter deposted in 18 test areas in Chicago and were incorporated into the model with the average accumulation of D/D and BOD used for parks directly and undeveloped land.

In using these data it is first necessary to estimate the number of feet of gutter per acre. This is best done from a map, but if none is available, a reasonable estimate may be made based on the size of the blocks. For example, blocks 330 feet square, or 220 feet by 500 feet, would have approximately 465 feet of gutter per acre. Although this size is small for Chicago, it may be a fairly good average, except where lot sizes are larger than 1/4 acre and blocks are unusually long.

The amount of this material on the ground at the start of the rain will depend on the frequency and efficiency of street sweeping methods as well as on the number of dry days since the last rain. At present the following procedures are being used to allow for these effects: if the number of dry days since the last rain is less than or equal to the frequency of cleaning, the D/D accumulation per day from the table is multiplied by the number of dry days to find the accumulation on the ground at the start of the rain. This procedure would appear to err on the high side, since it tacitly assumes that the streets are not swept in the interval, but it is probably on the low side during weeks when it rains every day or every other day, since the APWA study found very little effect on the accumulation of D/D due to rainfall between street cleanings (Ref. 2, p. 42). For a series of computer runs covering several months' or a year's rainfall the high and low estimates will probably average out. In attempting to check the results in a particular storm preceded by only one or two dry days it may be necessary to determine, if possible, when the streets were cleaned, or to assume a minimum number of days' accumulations, or to make allowance for the D/D not washed off by the preceding rains. A case in point is the storm of November 15, 1967, on the Laguna Street area in San Francisco (Ref. 3), which will be discussed later in this section.

If the number of dry days since the last rain exceeds the cleaning frequency, the equivalent number of days of accumulation is computed by the following formulas:

$$\text{NCLEAN} = \frac{\text{DRDAY}}{\text{CLFREQ}} \tag{6}$$

$$\text{TOTDD} = \text{CLFREQ} \times \text{DD} \times \left[1 + (1 - \text{REFF}) + \ldots + (1 - \text{REFF})^{\text{NCLEAN}}\right]$$

where DRDAY = Number of dry days before storm event

CLFREQ = Number of days between street cleanings

TOTDD = Pounds of dust and dirt on the ground before the start of storm

DD = Dust and dirt accumulation rate per subarea

REFF = Efficiency of street cleanings (see Table 11-3)

For example, in the case of the March 10, 1967, storm on the Laguna Street area in San Francisco, the number of dry days preceding the storm was 40, the cleaning frequency was 7 days, and the efficiency of cleaning was assumed to be 75 percent, resulting in an equivalent accumulation of 9.33 days.

The cleaning efficiency, REFF, is assumed to depend on both the number of passes of the sweeper and the frequency of cleaning in accordance with Table 11-3, which is based on a table in the APWA study and has been incorporated in the model.

Table 11-3. EFFICIENCY OF STREET SWEEPING IN CHICAGO AREA

| Cleaning Frequency | Efficiency as a Decimal Fraction<br>1 Pass | <br>2 Passes | <br>3 Passes |
|---|---|---|---|
| 15 days or more | .60 | .88 | .98 |
| 8 to 15 days | .70 | .92 | .98 |
| 7 days or less | .75 | .95 | .98 |

Source: APWA, "Water Pollution Aspects of Urban Runoff," January 1969, WP-20-15 (Ref. 2, p. 147).

## Tests Indicate Modifications Necessary

When $P_o$ was estimated directly from the APWA results and applied to the Cincinnati area without modification of the theory, it became evident immediately that the suspended solids were 5 to 10 times too high and the BOD was much less than reported. Preliminary trials using San Francisco data from Ref. 3 gave similar results.

This was temporarily disconcerting but eventually beneficial because it led to a more detailed study and deeper understanding of the APWA report. The effort resulted in modifications of the theory which have been checked satisfactorily against data available for the Laguna Street combined sewer in San Francisco (Ref. 3). In arriving at the necessary modifications, data in the Cincinnati study were used to great advantage.

## Availability Factor, A

Attention was directed primarily to suspended solids because of the very large differences encountered, and the conclusion was reached that not all the dust and dirt was normally available for the production of suspended solids.

That portion of the street litter passing a 1/8-inch hardware cloth was defined as the dust and dirt portion in the APWA study. Sieve analyses were made of dust and dirt removed by hand sweeping and by various machine methods and are reported in Appendix E of Ref. 2. Screen sizes were 10, 16, 20, and 30-mesh, so all that was measured was the grit component. The amount passing the 30-mesh sieve varied from 0.62 percent to 13.6 percent in 4 samples (one additional sample gave 42.06 percent) removed by machine sweeping; from 58.8 percent to 82.5 percent in 4 samples removed by hand sweeping; and from 65.48 percent to 93.18 percent in 4 samples removed by vacuum cleaning following machine sweeping.

Assuming a 70 percent efficiency for machine sweeping, it can be inferred from these data that approximately one-quarter to three-quarters of the dust and dirt is finer than 30-mesh, and that machine sweepers remove about 85 percent of the coarser material but a very much smaller proportion of the finer material. It is the finer material that contributes to the suspended solids load. The coarser material, if it gets into the sewers, constitutes grit, and, while its removal is essential in sewage treatment plants, it amounts to less than 2.5 percent of the suspended solids (Ref. 2, p. 111). These facts, plus the fact that the occurrence of rainfall between cleanings had little effect on the amount of dust and dirt removed (Ref. 2, p. 42), confirm the foregoing conclusion that only a portion of the dust and dirt is normally available for the production of suspended solids.

There is considerable information on dust fallout in the APWA study. Dust fallout on the Cincinnati area was also reported. It is stated

(Ref. 2, p. 105) that dustfall particulates vary in size from 1 to over 100 microns, but from another statement (p. 23) it can be concluded that the bulk of dustfall particulates range in size from 20 to 40 microns (.02 to .04 mm). Note that a 400-mesh screen has an opening size of 37 microns, and that 100 microns lies between 100- and 200-mesh. Grit chambers are usually designed to remove material coarser than 65-mesh, and in extreme cases coarser than 100-mesh. Dustfall that is washed into the sewers appears as suspended solids.

The following significant figures on dust fallout are contained in the APWA study (Ref. 2, pp. 23-25):

| Yearly Average | Tons/sq mi/mo |
|---|---|
| 1954-65 | 54.9 |
| 1965 | 41.0 |
| 1966 | 36.9 |

The range in measurements at 20 stations in Chicago in 1966 was from 20.6 to 61.4 tons/sq mi/mo. The dustfall on the Cincinnati area was reported as 506 lb/acre/yr, which is equivalent to 13.5 tons/sq mi/mo, or less than the lowest Chicago reading. Figure 2 (Ref. 2, p. 25) shows the dustfall on a 10-acre unit area varying throughout the year from 0.30 ton/mo to 0.79 ton/mo. The average of the seven summer months is 0.50 ton/mo or 3.3 lb/acre/day compared with a yearly average of 0.58 ton/mo. The corresponding figures at the minimum and maximum Chicago stations are 1.84 and 5.5 lb/acre/day. The Cincinnati figure, unadjusted for monthly variations, equals 1.38 lb/acre/day.

The weighted average D/D from Table 11-2, assuming 364 feet of curb per acre based on normal Chicago residential blocks 330 feet by 660 feet with no curb allowance for alleys, equals 1.5 x 3.64 or 5.46 lb/acre/day. The average summer dustfall of 3.30 lb/acre/day amounts to 60 percent of this figure. However, since the average imperviousness of the typical 10-acre unit area is 36 percent, of which 46 percent is roof area and 54 percent streets and alleys, the amount of dustfall in the D/D samples is 60 x .36 x .54 or 11.7 percent. It would be somewhat more if the wind blows dust off the roofs and trees into the street. An additional 10 percent settles on roof areas originally and may remain there until flushed off by rain or blown onto streets or grassed areas.

A theoretical study of overland flow from pavements combined with Shields' equation for sediment transport (Ref. 4) indicates that the closer a particle is to the gutter, the smaller the runoff rate necessary to transport it there, and the larger the particle, the greater the runoff rate required. This study indicated that a rainfall rate of .14 inch per hour would begin to move 20-micron material, and a rate of .65 inch per hour would begin to move 40-micron material. The action of traffic has a tendency to concentrate the dirt and dust as well as the street litter in or close to the gutter. Once in the gutter the finer material is readily transported. Using the equations for flow in gutters in Tholin and Kiefer's paper (Ref. 5), it was found that a runoff rate of .01 inch per hour would transport 20-micron material, and a rate of .03 inch per hour would transport 40-micron material. The overland flow equations indicate that very large rates of runoff would be required to remove dust and dirt from grass plots, and that unless erosion takes place from ungrassed areas, the contribution of pervious surfaces to suspended solids content is minor. However, the runoff from pervious surfaces may contain significant amounts of soluble pollutants.

The above analysis indicates that at the beginning of a rain a small proportion of the dust and dirt is immediately available, and that as the rain increases, more and more dust and dirt, represented by larger particles and particles farther from the water channels, becomes available.

Figure 9 of Ref. 1 shows the suspended solids concentration plotted against the mean flow of increasing flow ranges to fall on a straight line which can be represented by the following equation:

$$SS(\mathrm{mg/L}) = 100 + 650 \left(\frac{\text{runoff in cfs}}{8.36}\right) \qquad (7)$$

Since the drainage area is 27 acres, this equation transforms into the following for runoff $r$ in inches per hour:

$$SS(\mathrm{mg/L}) = 100 + 2{,}100\, r \qquad (8)$$

Since the area is served by a separate storm system, complications due to sanitary flow are avoided.

Introducing an availability factor A into the foregoing theory leads to the following equation for suspended solids:

$$SS(\mathrm{mg/L}) = \frac{AP_o(1-e^{-4.6\,rt})}{\text{area} \times r} \times \frac{268}{60t} \qquad (9)$$

Using a time-step of 10 minutes, or $t = 1/6$ hr , values of suspended solids were determined by both equations for each of the plotted points of Figure 9 in Ref. 1 and equated to determine values of $AP_o$ . A figure of 2,510 pounds was estimated for $P_o$ , equivalent to 93 pounds per acre, based on the figures in Table 11-2, assuming the land use to be 50 percent commercial and 50 percent single-family residential, 465 feet of curb per acre, and 10 antecedent dry days without street sweeping in the interim, i.e., 10 days accumulation of dust and dirt.

Using this value of $P_o$ allowed the computation of values of $A$ which were plotted against $r$ . The following equation of several tested fitted the data best and was adopted initially:

$$A = .057 + 1.4\, r^{1.1} \qquad (10)$$

The maximum computed value of $A$ was .44 for $r = .31$ . Based on the foregoing qualitative discussion of the amount and availability of the finer portions of the D/D, a maximum value of .75 is suggested for $A$ except where soil erosion is a factor, and is incorporated in the model.

The equation was first applied to the Cincinnati area for the storm of March 16, 1963, using $P_o = 2{,}510$ pounds. In the first application of the availability factor, a new value of $A$ for each time increment was multiplied by 2,510 pounds, and the sum of the suspended solids removed in all previous time increments was deducted from this product to obtain the amount available at the beginning of each time-step. The concentration of suspended solids in each time-step, as well as the plain and weighted averages, were plotted against the runoff in cfs. Only the initial points of low runoff lay on the curve of Figure 9 in Ref. 1, and both averages lay below it, indicating that the available solids were decreasing too rapidly as the computation proceeded.

A second computation was made in which the pounds removed in each time increment were deducted from D/D on the ground at the beginning of the time increment before multiplying by A, to obtain a new amount available, or REMDD, the D/D remaining on the ground. The computed results were much closer to the curve of Figure 9 in Ref. 1, and the plain and weighted averages straddled the curve. This method of computation was therefore adopted for further testing and is incorporated in the model.

A third method of computation similar to the first method was tried using a revised equation for A which was modified so as to make the computed points fit the curve of Figure 9 in Ref. 1 better, but instead of deducting $\Sigma$DD/dt from A times initial $P_o$, the amount removed up to the beginning of the time-step was recomputed as $\Sigma_o^r$ DD/dt, or what it would have been if the runoff had not exceeded the runoff in the time-step being computed. This procedure was based on the assumption that each increase in r and A involved larger or more remote particles that became available progressively as r increased, but that any portion available incrementally and removed for values of r in previous time-steps greater than r in the time-step under consideration, should not be deducted from A times initial $P_o$ since it is not contained in the new value of A. This procedure appeals theoretically, but the results were not significantly different from those of the second method and the computation procedure was a great deal more complicated. Therefore this method was not adopted.

All three methods of computation produced suspended solids concentrations of approximately the right order of magnitude, but none of them exhibited the pattern of decreasing concentration with length of storms as shown in Table IV of the Cincinnati study (Ref. 1). Since this table gives average results for a wide variety of storms, lack of agreement for a particular storm is not especially significant.

### BOD Modification

In tests of the method using the storm of March 16, 1963, on the Cincinnati area, the amount of BOD removed was approximately half of the annual average BOD in the storm runoff on a lb/acre/day basis, computed from Table 11-1, and the concentrations in mg/L were approximately half of those given in Table IV of Ref. 1. Initial attempts to correct this discrepancy in checking against Laguna Street data required adding the equivalent of 5 percent of the suspended solids removed to the BOD removed on the assumption that this was nonsoluble BOD not included in the analysis for BOD in the APWA study (Ref. 2). The addition of 10 percent was required in checking Selby Street, San Francisco. A closer look, however, at the method of anlaysis used in that study does not indicate that the BOD tests were run on filtered samples; therefore the figures given in Table 11-2 should include both soluble and nonsoluble BOD. The APWA figures would not include BOD due to leaves, grass, or organic material not passing the 1/8-inch hardware cloth used to screen out the coarse solids. It is also evident that the BOD figures in the table cannot include BOD in the drainage from roofs, areaways, parking lots, grassed areas, and garden plots. The latter areas produce soluble

BOD from decaying vegetation, fertilizer applications, and animal excreta. In Table 11-1, taken from the Cincinnati study, the BOD in the surface runoff amounts to 4-1/2 percent of the suspended solids load, as against an average of 5 mg per gram of dust and dirt on the streets from the APWA study. Until further data become available during field checking of the program, it is proposed that a percentage equivalent of the suspended solids removed in each time-step be added to the BOD obtained by application of the procedures outlined herein. It is anticipated that the percentage may range from 3 to 10 percent. The current model includes a figure of 5 percent.

## BOD from Catchbasins

Catchbasins traditionally have been built on inlets to combined sewers and storm drains for the purpose of removing heavy grit and detritus which might otherwise settle out in the piping system. The construction provides a trapped pocket of liquid and solids in which the organic component undergoes decomposition between rains. The APWA study (Ref. 2, p. 85) listed the BOD content of 7 catchbasins, sampled after several days without rain. The BOD varied from 35 to 225 mg/L at 5 commercial locations and from 50 to 85 mg/L at 2 residental locations. Limited sampling of 11 catchbasins in Washington, D.C., during storms was carried out over a period of one year (Ref. 6). The BOD ranged from 6 to 625 mg/L and averaged 126 mg/L. Palmer (Ref. 7) sampled catchbasins in Detroit in 1949, reporting an initial BOD of 234 mg/L in the business district decreasing to 96 mg/L after 2-1/2 hours. These data show that catchbasins constitute an important source of pollution. The data quoted above for Washington and Detroit include the BOD of the surface wash flowing through the basins and do not correlate concentration with runoff rate.

In order to determine how the pollution load in catchbasins at the start of a storm is flushed into the sewers, the APWA study added 15 to 45 pounds of salt dissolved in water to a catchbasin containing 353 gallons. Water from a hydrant was discharged through a hose and water meter to the gutter adjacent to the catchbasin. Samples were taken when various total quantities up to 1,685 gallons had been added to and passed through the basin. A stirring device ensured thorough mixing of the samples, which were analyzed for chloride content. From the results the cumulative percent of salt discharged and by inference the percent of pollution discharged was plotted against gallons of liquid added (Ref. 2, Figure 9, p. 88).

The following empirical equation has been developed to fit the curve of this figure:

$$R = 100(1.0 - e^{\frac{-x}{1.6v}}) \qquad (11)$$

where R = Percent of catchbasin source pollution removed

x = Cumulative inflow to catchbasin (gal.)

v = Trapped volume of liquid in basin before storm (gal.)

The number of catchbasins per acre, the assumed BOD concentration in mg/L, and the average volume of liquid in the basins below the overflow level are read into the program as data, and the pounds of BOD removed in each time-step are computed in accordance with the above equation and added to the BOD washed off the surface during each time-step. No adjustments are made as long as the antecedent dry period is at least a day, but if the period between storms is less than a day the model multiplies the initial pollution load by the square root of the length of the dry period expressed as a fraction of a day. Thus fat the effect of catchbasins on pollutant concentration has been confined to BOD.

## Displacement and First Flush Effects

The effect of the displacement or crowding of the dry weather pollution load in combined sewers towards the outlet due to the increase in velocity in the sewers with increasing storm flow was satisfactorily modeled by the Transport Model. It also takes account of deposition and scour in the sewers where it is a factor.

## Coliforms in Surface Runoff

The method used to model coliforms in surface runoff is again based on APWA (Ref. 2, p. 55) data related to studies conducted in Chicago. The APWA study reported that the confirmed total coliforms per gram of dust and dirt swept from the streets were of the following magnitudes:

| | |
|---|---|
| Single-family residential areas | $1.3\times10^6$ |
| Multi-family residential areas | $2.7\times10^6$ |
| Commercial areas | $1.7\times10^6$ |

Using these values, with appropriate unit conversion, provided a simple estimate of total coliforms available in each subcatchment and, by direct relationship with the suspended solids, the amount washed into storm water inlets during each time-step of a storm event.

## TEST APPLICATION

As a test of the method it was applied to the Laguna Street sewer in San Francisco. This is a combined system draining a steeply sloping area of 370 acres, mostly apartment houses, with a population density of 68 persons per acre. The drainage area rises to an elevation of about 300 feet above the outlet, and the trunk sewer, which is 8,600 feet long, contains some relatively steep grades. Ref. 3 contains measurements of flow and quality, including suspended solids and BOD, for the storms of March 10 and March 15, 1967.

The storm of March 10 was preceded by an antecedent dry period of 40 days, except for .01 inch of rain on 4 days and .04 inch on one day in this period which were considered small enough to ignore. As mentioned earlier, using a street sweeping frequency of 7 days and an efficiency of 75 percent, the accumulation of D/D on the ground at the start of the rain was equal to 9.33 days.

Since the sewer system is of the combined type it was necessary to allow for the suspended solids and BOD of the dry weather flow. The BOD contributed by the catchbasins was also included. Furthermore, due to the status of program development at the time these checks were made, it was necessary to consider the area as a whole, instead of being made up of a number of subareas which are routed through the Transport Model and combined with other areas as the routing proceeds downstream. To allow for the possible effect of this difference a displacement effect was temporarily included in the checking program to substitute for the effect of the Transport Model. All these allowances had very little effect on the suspended solids but a considerable effect on the BOD. Figure 11-1 shows the comparison with the reported results. In view of the differences between the cities of Chicago and Cincinnati, which furnished the basic data for the method, and San Francisco, the agreement is remarkable. The agreement at peak flows is better than at the low flows, which means that the total pounds of pollution (the product of flow and concentration) are being modeled satisfactorily. A similar comparison between the reported and computed total coliforms is shown in Figure 11-2.

The storm of March 15 was also used as a check, considering suspended solids only, and neglecting any contributions or effects of the dry weather flow. The antecedent dry period was assumed to be 5 days. This storm began about 8:20 p.m. with a light rain which increased the overflow rate from 7 to 12.5 cfs at 8:30 p.m. The flow then steadily decreased to 2.0 cfs at 9:30 p.m., followed by a large increase to 108 cfs at 10:30 p.m. It dropped to 11.4 cfs at 11 p.m. and then rose to a secondary peak of 76 cfs at 11:15 p.m., which decreased to 44 cfs by midnight. The method successfully modeled the suspended solids level at the first high peak at 10:30 p.m. and at the preceding and following low flows. It was low on the first minor rise at 8:30 p.m. and high on the second high peak at 11:15 p.m. It was later discovered that significant rain occurred on four of the preceding five days, as the records from the Federal Building show in the list which follows.

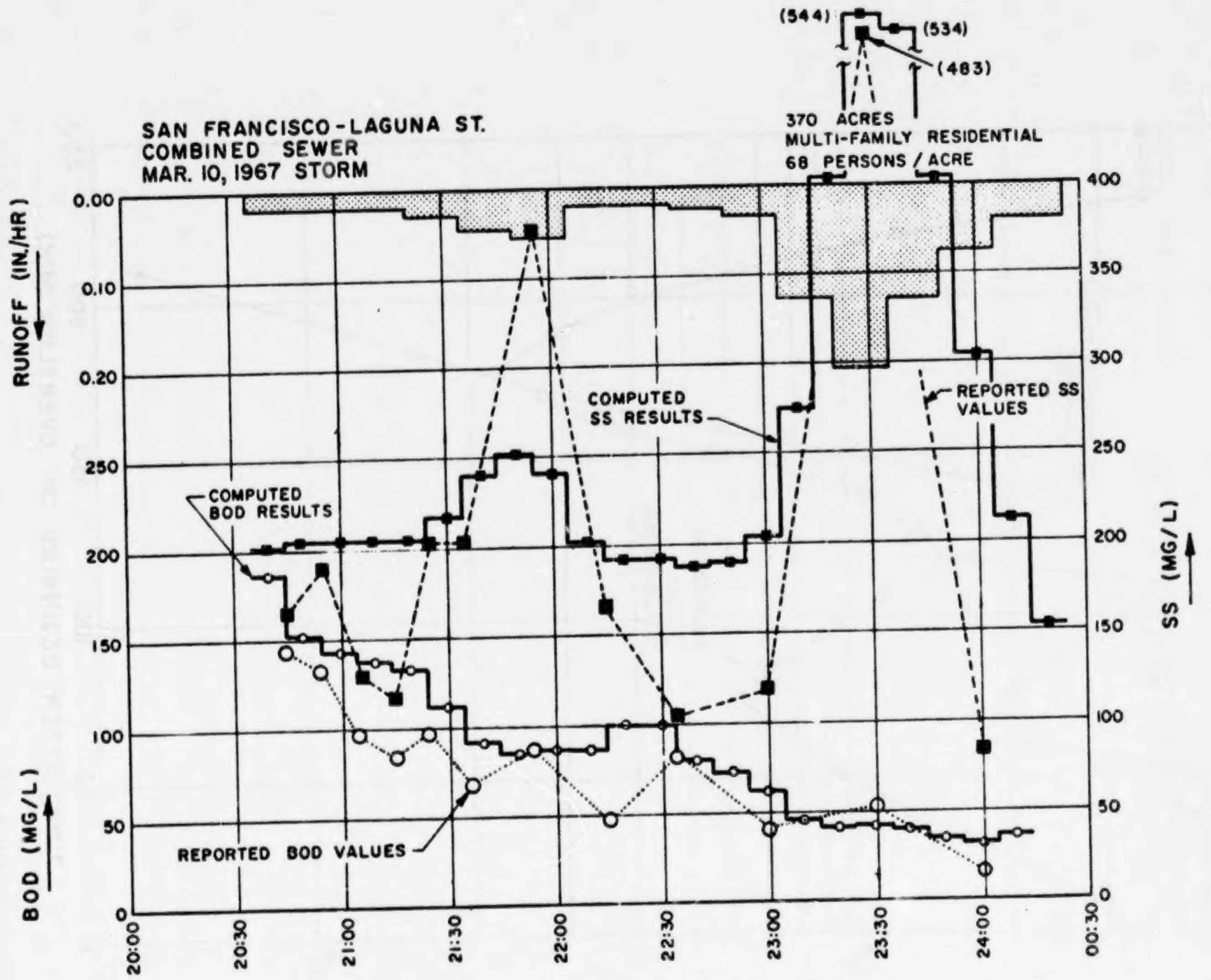
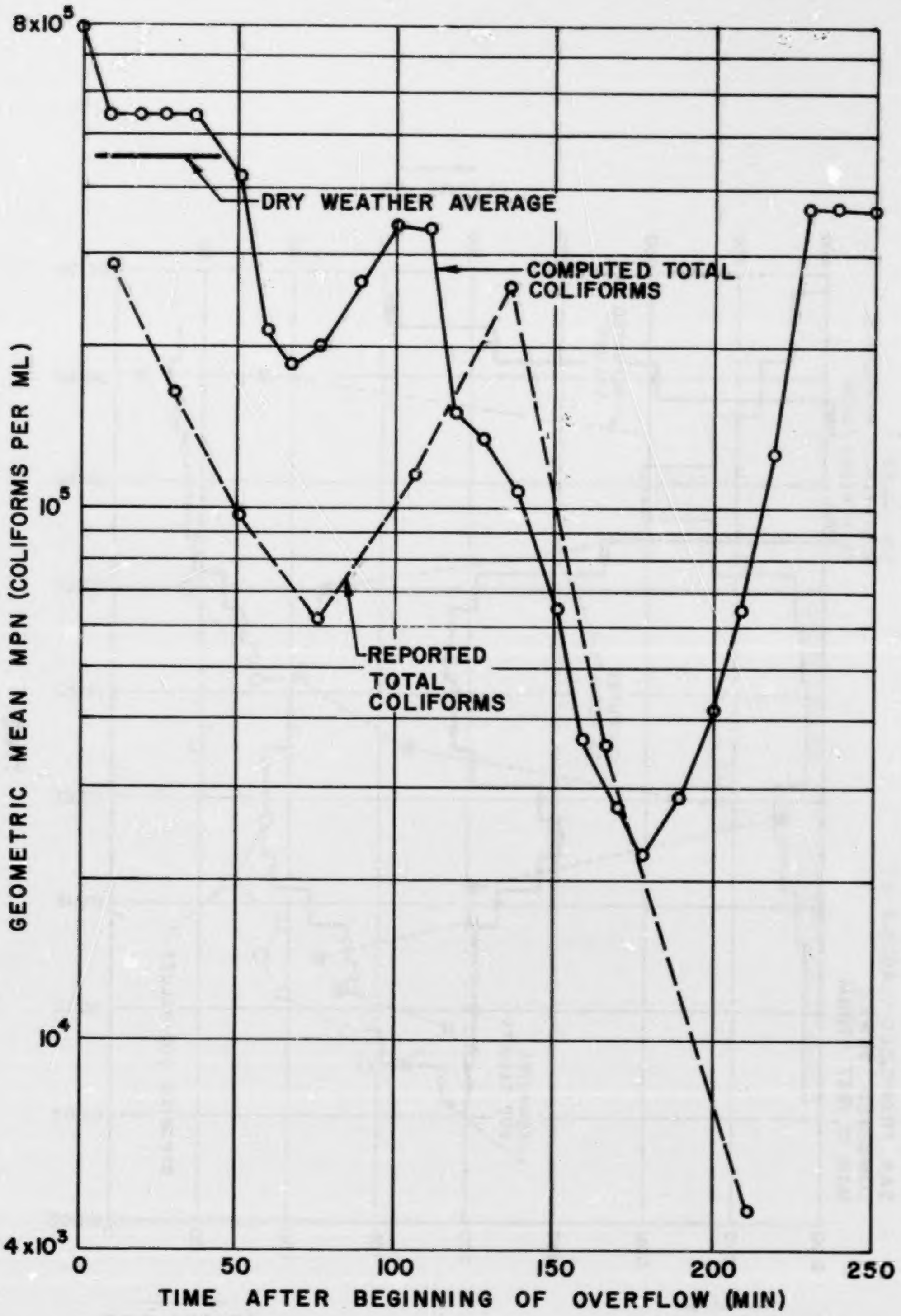


Figure 11-1. BOD AND SS TEST RESULTS FOR COMBINED SEWERS, LAGUNA STREET, SAN FRANCISCO



San Francisco-Laguna St.
Combined Sewer March 10, 1967 Storm

Figure 11-2. TOTAL COLIFORM TEST RESULTS FOR COMBINED SEWERS

| Date | Rain (in.) |
|---|---|
| March 10 | .49 |
| March 11 | .45 |
| March 12 | .58 |
| March 13 | .50 |
| March 14 | .03 |
| March 15 | .43 |

The rain on these days was all of the same order of magnitude as the rain on March 10 during which storm 16 percent of the D/D initially on the ground was removed. If it is assumed that the streets were swept on March 11, and that each day's rain except the sprinkle on March 14 removed 16 percent of the D/D on the streets after allowing for the addition of that day's contribution, there would have been an accumulation of 4.84 days on the street at the beginning of the rain on March 15. This analysis may justify the above described check based on 5 days. It also indicates that in such cases it may be desirable to assume a minimum number of antecedent dry days, say 3 to 5 days, regardless of the weather pattern.

# SECTION 12

## DRY WEATHER FLOW QUALITY MODEL

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# SECTION 12

## DRY WEATHER FLOW QUALITY MODEL

### OBJECTIVES

The objectives of the DWF Quality Model are to:

1. Provide reasonable estimates of the modeled quality constituent concentrations based on non-storm influences.
2. Function compatibly with other modeling subroutines with emphasis on time and spatial variations.
3. Require minimal input data consistent with desired results.

### INITIAL CONCEPTS

One way to establish a valid estimate of DWF quality is to sample and analyze it. For larger systems such sampling may be accomplished on a routine basis at the municipal sewage treatment facility. Since these data would normally, or most economically, be available at only one or a few points in the prototype system, a means to extrapolate or apportion results over the entire system is required.

A second approach for estimating DWF quality is to assume average per capita values and to compute results on the basis of population distribution.

These concepts, coupled with variations according to flow source (residential, industrial and/or commercial), family income, and other recorded parameters, form the basis for the model development.

## THE MODEL SUBROUTINE

The DWF Quality model is embedded within subroutine FILTH in the Transport Block. Subroutine FILTH also computes the DWF quantity estimate and provides a vehicle for adding infiltration (INFIL) so that both concentrations and mass values may be determined.

Since the Transport Model operates on a true simulation theory--i.e., pipe configurations and flow source locations are critical--DWF computations are designed to yield representative flows at several geographically distributed entry points.

The sum of all entry point values is, of course, the total DWF of the system.

Furthermore, if the entry point values are summed in sequence starting with the most upstream elements, valid subtotals may be printed out at intervals along the main trunk sewer for intermediate checks.

Once picked up by the Transport Model the DWF is mixed and routed in a manner identical to the storm water routing, discussed elsewhere.

## THEORETICAL DEVELOPMENT

Dry weather flow (that flow measured in combined sewer systems between storm events) is assumed to be divisible into the following four categories:

Domestic flow

Commercial flow

Industrial flow

Infiltration.

## Domestic Flow

The domestic flow represents the flow from the homes of all residents of an area whether they be barracks, apartments, multi-family, or single-family dwellings.

The first concern in computing domestic flow quality is a population estimate, which for most urban areas is readily available in Census Tract data tabulations. First approximations as to the quality makeup may use average values in lb/capita/day or similar units taken from standard sanitary engineering textbooks, such as Fair and Geyer (Ref. 1).

Next in importance is an estimate of the percentage of households equipped with garbage grinders, as these units appreciably increase the waste concentrations in the flow. A sample comparison of ground garbage with sewage is shown in Table 12-1 as taken from Haseltine (Ref. 2).

Finally the strength of the wastewater flows, as well as volume, may vary with family income as shown in Table 12-2 taken from Watson (Ref. 3).

## Commercial Flow

Flows from areas zoned for commercial use may be expected to include domestic flow from the overlapping resident population plus flows from stores, offices, and other business units (laundromats, movies, hotels, etc.). Hubbell (Ref. 4) found these flows generally similar to residential area flows. In the same work, Hubbell also reported hospital, resident, school, and institutional flows to be generally stronger (lb/capita/day) than "normal" domestic flow. Because hospitals, schools,

Table 12-1. COMPARISON OF QUALITY CONSTITUENTS OF GROUND GARBAGE WITH SEWAGE

| | Ground Garbage, lb/capita/day | Sewage, lb/capita/day |
|---|---|---|
| Total solids | 0.15 | 0.55 |
| Total volatile solids | 0.13 | 0.32 |
| Suspended matter | 0.10 | 0.20 |
| 5 Day BOD | 0.08 | 0.17 |
| Fats and greases | 0.03 | 0.05 |
| Nitrogen | 0.002 | 0.04 |
| Moisture | 0.45 | ---- |

Source: T. R. Haseltine, "Addition of Garbage to Sewage," 1950 (Ref. 2).

Table 12-2. COMPARISON OF QUALITY CONSTITUENTS WITH FAMILY INCOME

| Item | Home 1 | Home 2 | Home 3 |
|---|---|---|---|
| Approximate market value | $45,000 | $25,000 | $18,000 |
| Total number of people | 5 | 5 | 5 |
| Water consumption, gal./capita/day | 78 | 66 | 24 |
| Suspended solids, lb/capita/day | 0.23 | 0.16 | 0.10 |
| COD, lb/capita/day | 0.45 | 0.30 | 0.18 |
| $BOD_5$, lb/capita/day | 0.35 | 0.16 | 0.10 |
| Grease, lb/capita/day | 0.06 | 0.18 | 0.13 |

Source: K. S. Watson et al., "The Contribution from the Individual Home to the Sewer System," 1967 (Ref. 3).

and institutions are better represented as point rather than area sources of waste, they are most easily modeled as special cases of industrial flow discussed below. Selected findings by Hubbell are reported in Table 12-3.

Table 12-3. ESTIMATED TYPICAL WASTEWATER FLOW AND CHARACTERISTICS

| Source | Flow, gal./capita/day | BOD, lb/capita/day | SS, lb/capita/day |
|---|---|---|---|
| Suburban residential | 75 | 0.17 | 0.20 |
| Hospitals | 175 | 0.25 | -- |
| Factories | 20 | 0.10 | 0.10 |
| Motels | 50 | 0.17 | 0.20 |
| Shopping centers | 60* | 0.18* | 0.18* |
| Schools, non-resident | 10 | 0.03 | -- |
| Schools, resident | 75 | >0.17 | >0.20 |
| Institutions | 120 | 0.31 | 0.31 |

*Per employee.

Source: J. W. Hubbell, "Commercial and Industrial Wastewater Holdings," 1962 (Ref. 4).

Industrial Flow

Flows from areas zoned for industrial use, like commercial areas, may be expected to include domestic flow from overlapping resident population and domestic-type wastes from employees. Most important, however, are the process flows (wastes generated by wet industrial processes), which vary with the type of product produced and rate of production less removals accomplished prior to discharge (pretreatment). Measured characteristics are used to the extent available in model applications and are supplemented, when necessary, by generalized data, such as may be found in Rudolph (Ref. 5). In modeling, one data card is required for each significant industrial process flow source describing its average daily flow and constituent quality concentrations.

### Infiltration

Infiltration is assumed to be pure (free of contaminants) unless known to be contaminated by passage through soils having significant soluble impurities of the type being modeled.

### Computations

Computations proceed along one of two paths depending upon whether measured characteristics are available (Case 1), or whether average per capita values must be assumed (Case 2). In each case the quality constituents modeled are BOD, total suspended solids, and total coliforms.

Case 1. In this situation the total quantity and characteristics of the average DWF are known, either from direct sampling and analyses or available treatment facility data, and the quantities and distribution of DWF and infiltration are known from previous modeling routines. The proper distribution of the known waste characteristics is accomplished by first deducting the non-domestic flows and characteristics from the total values, following the equation:

$$\begin{pmatrix}\text{total flow and}\\ \text{characteristics}\end{pmatrix} - (\text{infiltration}) - \begin{pmatrix}\text{process flows and}\\ \text{characteristics}\end{pmatrix} = \begin{pmatrix}\text{domestic flows and}\\ \text{characteristics}\end{pmatrix} \tag{1}$$

Further refinement in the estimate of domestic flow and characteristics is made by weighting the values according to the fraction of commercial flow, high income residential flow, average income residential flow, low income residential flow, and the percent usage of garbage grinders.

For BOD and suspended solids the base average quality characteristics are then determined from the weighted estimates in terms of lb/day/cfs of flow. The base average for coliforms is simply the measured value, MPN/day, divided by the total population.

These base averages are then coupled with the appropriate modifiers (income, commercial use, garbage grinders, or population) and flow to reconstruct the DWF and characteristics for each subarea within the system. Process flows, characteristics, and infiltration are likewise added at the appropriate geographical location.

Case 2. In this case the procedure is similar to Case 1 except that the base average quality constituents are computed directly, based on average textbook values or figures from earlier model runs.

Hourly Variation

The results of the foregoing computations yield average daily values. Since both flow volumes and quality characteristics change throughout the day (Refs. 6,7), hourly variation factors are incorporated to adjust these averages to real time conditions corresponding to the start and duration of the storm event.

Further discussion and explanation of the procedures are given in the "User's Manual."

## Default Values

Default values used in programming the model are built in as follows:

MODIFIERS

| Item | Value |
|---|---|
| Commercial flows | 0.9 |
| Low family income, less than $7,000 per yr | 0.8 |
| Average family income, $7,000-15,000 per yr | 1.0 |
| High family income, greater than $15,000 per yr | 1.2 |
| Garbage grinders in use (in addition to income modifier) | 1.3 |

AVERAGE QUALITY CONSTITUENTS (Case 2 only)

| Item | Value |
|---|---|
| BOD, average family income | 0.20 lb/capita/day |
| SS, average family income | 0.22 lb/capita/day |
| Total coliforms (Ref. 1) | 200 billion/capita/day |
| Flow without infiltration | 85 gal./capita/day |
| Flow with infiltration | 100 gal./capita/day |

Family incomes are based on reported 1960 census data. The table values were selected as being somewhat credible and can readily be changed for specific applications. Hourly variation factors for flow and quality are considered too important to be locked into the program and thus must be fed in as basic data for each new system.

## TEST APPLICATIONS

The lack of data reporting DWF characteristics measured simultaneously at several points in a conveyance system precluded any comprehensive test applications. Good DWF quality monitoring results for single points in two San Francisco systems (Ref. 6), however, provided a limited base for model analysis.

The most significant outcome of these tests was the realization of the significance of the compounded factors for hourly variation. Measured values of both flow and quality variations showed the extreme variation of pounds of BOD released per 10-minute interval exceeded a ratio of 50 to 1 (largest 10-minute release divided by the smallest) in one day.

## CONCLUSION

A simple modeling technique has been devised to distribute quality constituents known at a single downstream point to their approximate source locations within a catchment area. This distribution is based on land use, population, family income, use of garbage grinders, and known or estimated process flows.

This breakdown is necessary to account properly for routing in the conveyance system during storm events and to support the Decay (sediment-scour) model discussed in Section 13.

Where quality constituents are not known, average values are provided for initial approximations.

Hourly variations in DWF quality constituents may exceed a range of 50 to 1 between extreme values in one day; thus the magnitude of DWF quality contribution to combined sewer overflows is heavily dependent on the real time occurrence of the storm event.

## SECTION 13

## DECAY MODEL

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# SECTION 13

## DECAY MODEL

### OBJECTIVE

The Decay model was developed to route pollutants and to account for the decay of pollutants during transit in a sewer system.

### THE MODEL SUBROUTINE

The computer model consists of two parts, a quality section (subroutine QUAL) and a dry weather load section (subroutine DWLOAD); both are incorporated into the Transport Model. The basic methodology developed for the Decay model is found in QUAL. An extension of this concept is the DWLOAD section.

In QUAL, mass balance and first order decay of pollutants provide a basis for the model. The general assumptions are that mixing within each sewer element in the system is instantaneous and complete, and that the nonconservative pollutants decay according to the first order reaction. An unidirectional transportation concept was used to describe the movement of pollutants through any specified sewer system, given sewer data and concurrent flows and velocities. The four quality parameters considered in the model are BOD, DO, suspended solids, and a conservative pollutant (P). Quality constituents such as coliforms are handled as a conservative pollutant. A sediment uptake and deposition phenomenon is conroproated into the model for the case of suspended solids.

Relative to the sediment uptake and deposition concept developed for QUAL, DWLOAD was developed to establish the initial sediment load within the sewer system. An iterative method is used to estimate daily sediment accumulation in each section of the sewer under DWF conditions. Also, using the assumption that the daily sediment build-up is constant over consecutive dry weather days, initial sediment load estimates for each conduit are made possible.

## THEORETICAL DEVELOPMENT

An investigation into the area of the decay of pollutants and sediment transport in sewers did not reveal any practical applications of extant theoretical developments and experiments. Thus, model development was geared toward a methodology that would encompass the major factors and information obtained from the literature search combined with present knowledge of river systems.

### Literature Survey

Further research is needed in the theory related to the decay of pollutants and sediment transport in sewer systems. Some doubt exists as to the significance of quality change, other than mixing, during rainfall-runoff periods. It has been suggested (Ref. 1) that the time of transit through the sewer system is too short to consider bacterial decay or reduction of BOD with time.

Decay of pollutants and sediment transport, as with infiltration, has a basis for theoretical analysis (Ref. 2). Raths and McCauley studied sediment transport in sewers (Ref. 3). The FMC Corporation also has conducted a study on periodic flushing and sediment transport (Ref. 4). Through these analyses, it is evident that particle size and specific gravity, depth of flow, and the slope of the conduit are important factors affecting deposition. Hill et al. (Ref. 5) in a recent study, investigated the flat bed phenomenon in alluvial channels. Also useful in developing a methodology for sediment uptake and deposition were the velocity relationships in Fair and Geyer (Ref. 6) and in Carstens' work on heterogeneous flow of solids in conduits (Ref. 7). Data needs relative to particle size were obtained from an APWA report (Ref. 8). Oxygen balance in sewers is described by Gustafsson and Westberg (Ref. 1). These authors applied the classic Streeter-Phelps equation (Ref. 9) to flowing sewage and solved for the oxygen uptake as a function of time, temperature, slope, and flow conditions in sewers.

Methodology

The general equation of the model representing continuity of mass is as follows:

$$\begin{array}{c}\text{Pounds in}\\ \text{element at}\\ \text{new time-step}\end{array} = \begin{array}{c}\text{Pounds in}\\ \text{element at}\\ \text{old time-step}\end{array} + \begin{array}{c}\text{Pounds}\\ \text{entering}\end{array} - \begin{array}{c}\text{Pounds}\\ \text{leaving}\end{array} \pm \begin{array}{c}\text{Pounds}\\ \text{decayed or}\\ \text{generated}\end{array} \pm \begin{array}{c}\text{Pounds entering or}\\ \text{leaving from}\\ \text{source or sink}\end{array} \qquad (1)$$

The mathematical rigor related to the development of the Decay model, using the concept of Eq. 1 and expanding it into its mathematical form, is given in Appendix B. Assuming complete mixing along with the use of a finite difference scheme, the final computer model for routing pollutants is in the form:

$$C_{n+1} = \left\{ C_n \left[ \overline{V}_n \left( \frac{2}{\Delta t} - (D_1 + D_2) \right) - (Q_{out})_n \right] + (C_{in} Q_{in})_n + (C_{in} Q_{in})_{n+1} + D_2 S (\overline{V}_n + \overline{V}_{n+1}) \right\} \Bigg/ \left\{ \overline{V}_{n+1} \left[ \frac{2}{\Delta t} + (D_1 + D_2) \right] + (Q_{out})_{n+1} \right\} \qquad (2)$$

where $C$ = Conc. of pollutant in element (lb/cf)

$\overline{V}$ = Volume in element (cf)

$n$ = Time-step number

$Q$ = Flow rate (cfs)

$D_1$ = Decay rate or oxygen utilization rate (1/day)

$D_2$ = Growth rate or reaeration rate (1/day)

$S$ = Maximum growth or oxygen saturation value (mg/L)

$\Delta t$ = Time increment (sec)

As mentioned earlier for the case of suspended solids, a sediment uptake and deposition model was incorporated into the Decay Model. When investigating deposition in sanitary sewers, Raths and McCauley (Ref. 3) claimed that consideration must be given to the importance of: type, size, and specific gravity of potential deposition material; quantity of flow; depth of flow; velocity of flow; pipe slope; pipe alignment; pipe size, pipe joints; pipe age; pipe roughness coefficients; and sewer construction practices. Thus, in developing the sediment model, the objectives were to relate the variables as closely as possible to the factors affecting deposition and to use the information available from existing models, such as the Transport Model.

The sediment model consists of two subroutines which follow similar mathematical procedures. First, DWLOAD determines the amount of sediment accumulated in each conduit since the last cleansing of the sewers. DWLOAD calculations are performed prior to flow routing, thus establishing initial sediment conditions in the conduits before actual routing. The next subroutine (ROUTE) determines the amount of sediment uptake and deposition during storm conditions. This subroutine is used continually during flow routing to determine $(C_{in})_{n+1}$. Eq. 2 is then used for routing the pollutant. Both subroutines utilize a procedure of solving for the particle diameter corresponding to the velocity in the sewer conduit. The governing equation in both cases is

$$d = \frac{RS'}{k(S_s-1)} \qquad (3)$$

where $d$ = Particle diameter (ft)

$R$ = Hydraulic radius (ft)

$S'$ = Conduit invert slope (ft/ft)

$k$ = Shield's k as criterion for deposition and resuspension

$S_s$ = Specific gravity of sediment

Using Eq. 3 as a basis for the model, the final equations resulting from the mathematical development of a sediment algorithm are

$$Sc \;=\; Sc + P \times (T_1 + T_2 + T_3) \times \Delta t \qquad (4)$$

$$(Css_{in})_{n+1} \;=\; \frac{\left[(1 - P) \times (T_1 + T_2 + T_3)\right]}{Q_{in}} + \frac{(1 - P) \times Sc}{\bar{V}_{n+1}} \qquad (5)$$

$$Sc \;=\; P \times Sc \qquad (6)$$

where Css = Concentration of suspended solids (lb/cf)

P = Fraction of sediment in suspension with diameter greater than or equal to the critical particle diameter

Sc = Amount of settled sediment in element (lb)

The process is that for each time-step the amount of sediment at the bottom of each conduit is determined by Eq. 4. Then, using Eq. 5, the amount of sediment to be routed is determined. This information is then used in Eq. 2. Finally, Eq. 6 updates the amount of sediment at the bottom of the sewer element for the next time-step. The detailed development of the model is presented ~~in~~ Appendix B.

## TEST APPLICATION

Sensitivity analyses were performed with the Decay model using data from the Selby Street, San Francisco, combined sewer system (Ref. 10) and the FMC Corporation study (Ref. 4). Results from the FMC sewer flushing project were used for a detailed investigation of the sediment uptake and deposition model.

## Selby Street, San Francisco

The main test areas used for these analyses are shown in Figure 13-1. Test Area 1 includes the first ten elements and Test Area 2 includes these ten elements plus the main trunk sewers in the system. In the simulation of conservative pollutants, the initial conditions for the sewer system were assumed at zero concentration. For nonconservative pollutants, i.e., BOD and suspended solids, initial conditions were set at DWF concentrations, but for DO, initial conditions were assumed at zero concentration.

Analyses were first performed for a mathematical breakdown of Eq. 2. These tests confirmed the existence of consistency in the continuity of mass and the complete mixing assumption. Conservative pollutants were then studied using Test Areas 1 and 2. The procedure was to check the system randomly at the elements to see if the total pollutant quantity input was equal to total output. These runs were successful using different pollutograph concentrations.

The next step was to study the model's reaction to different values of D1 and D2 in Eq. 2. The rate constant D1 was analyzed according to information obtained on the range of the rate constant for deoxygenation in raw sewage at 20°C (Ref. 1). Values used for the decay rate are in the range $0 \leq D1 \leq 3.0$. Results of the runs are shown in Figure 13-2. As anticipated, the analyses revealed that there is a dependency only on the duration and intensity of the storm and not on the pollutograph used. System response to the DO quality parameter also was tested for various hypothetical situations. Using D1, D2, and S, these results showed that

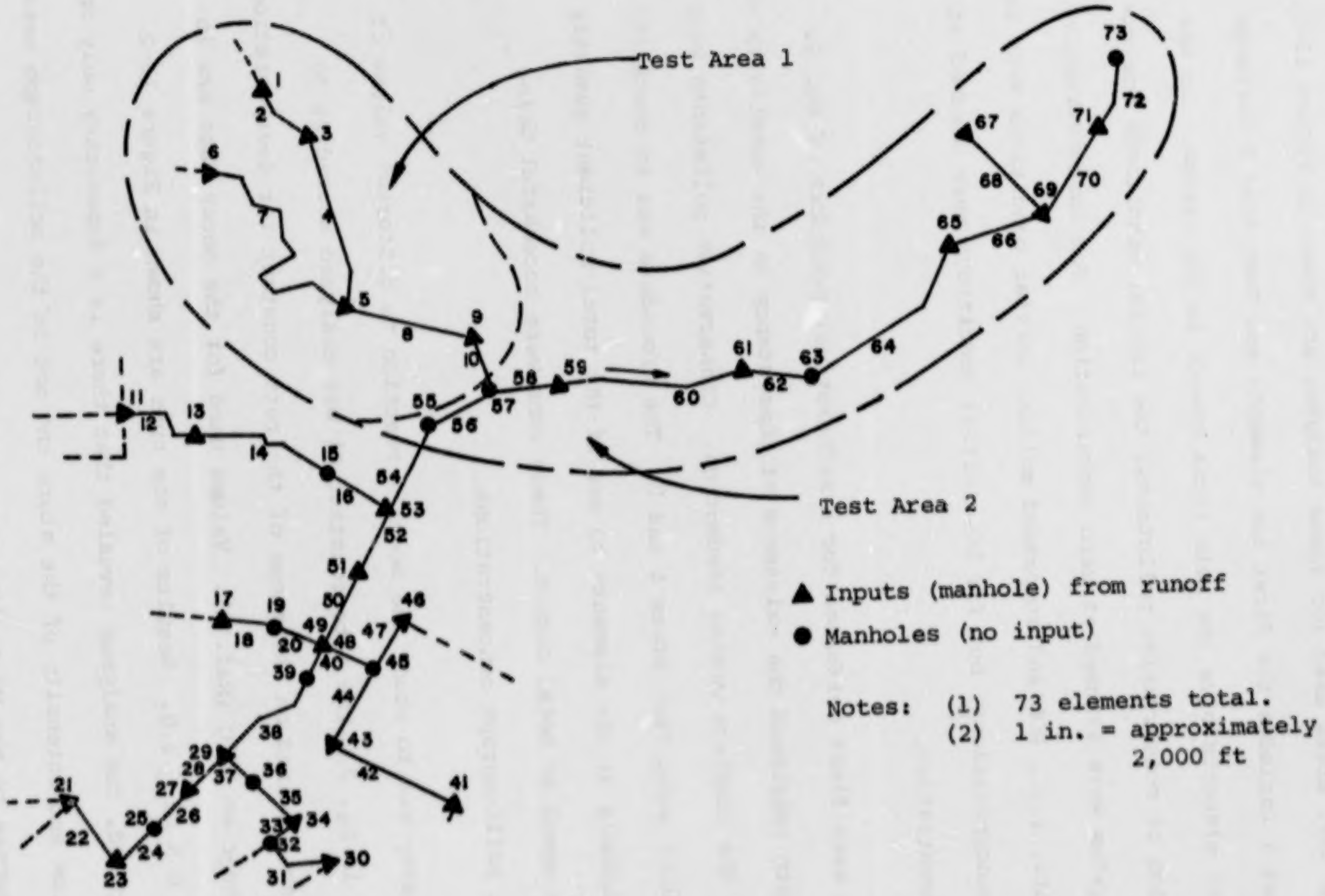
Test Area 1
Test Area 2
▲ Inputs (manhole) from runoff
● Manholes (no input)
Notes: (1) 73 elements total.
(2) 1 in. = approximately 2,000 ft
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42 43 44 45 46 47 48 49 50 51 52 53 54 55 56 57 58 59 60 61 62 63 64 65 66 67 68 69 70 71 72 73

Figure 13-1. SCHEMATIC DRAWING OF TEST AREAS, COMBINED SEWER (NUMBERED), SELBY STREET, SAN FRANCISCO

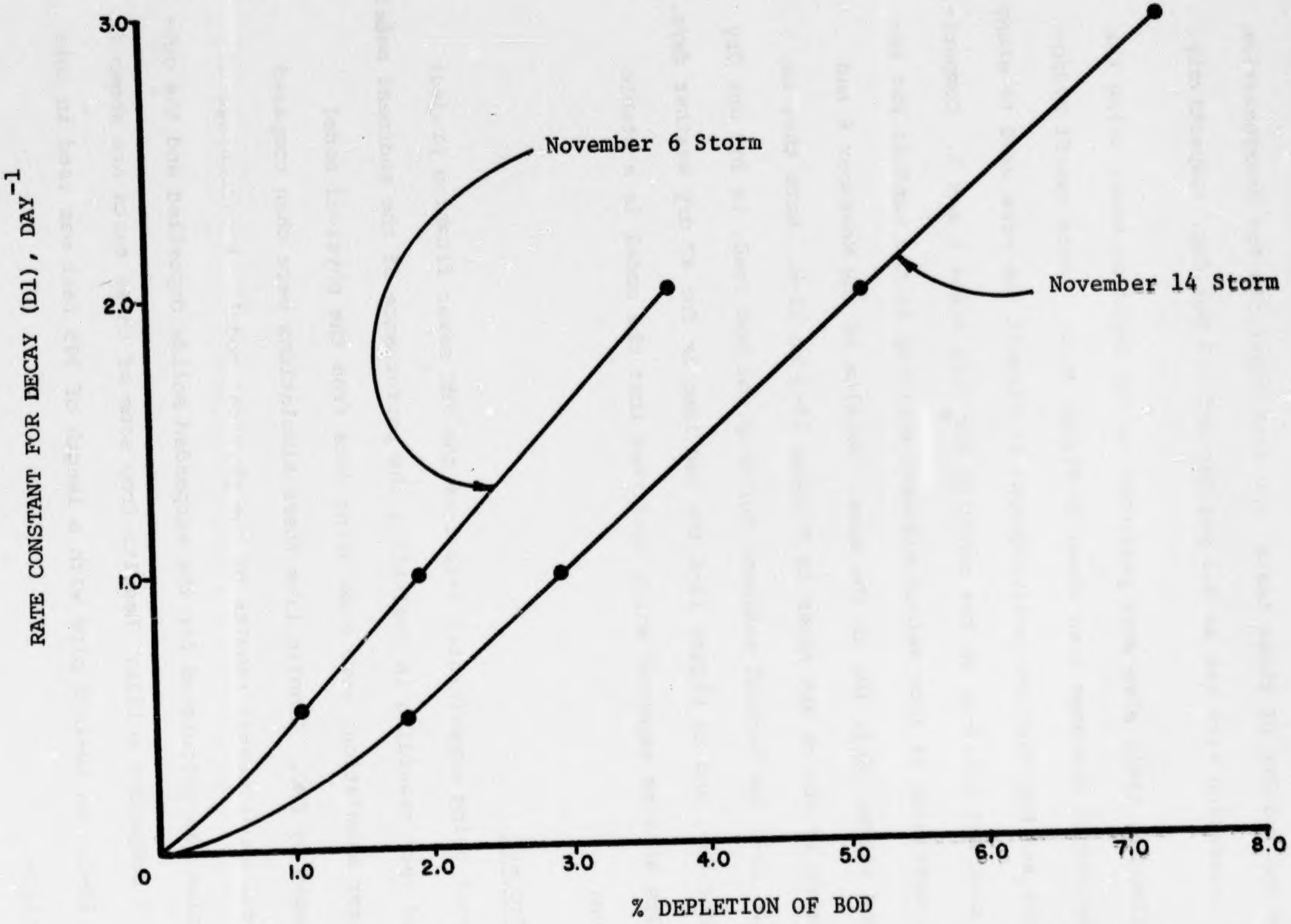

Figure 13-2. RESULTS OF SENSITIVITY RUNS FOR RATE CONSTANT FOR DECAY (D1), STORMS OF NOVEMBER 6 AND 14

reaeration of DO occurred and that the model was functioning as expected. From the results of these tests, the rate constants for deoxygenation and reaeration were set as 0.2 per day and 0.3 per day, respectively.

Preliminary tests also were performed on the sediment model using the Selby Street drainage area shown in Figure 13-1. Storm runoff hydrographs and hypothetical pollutographs at element one were used to study the sediment build-up in the conduits for Test Areas 1 and 2. Comparisons were made of flow versus sediment build-up in the conduit for recorded storms (Ref. 10) in the area. Results of the November 6 and November 14 storms are shown in Figures 13-3 to 13-5. Note that in Figure 13-3 the initial sediment build-up, or bed load, is for one dry weather day, and in Figure 13-4 the bed load is for 47 dry weather days. Results were as expected which confirmed that the model is a stable system.

## FMC Project

Analyses using experimental data from the FMC sewer flushing project proved very rewarding in evaluating the performance of the sediment model. Computer simulations were made using data from the physical model developed by FMC. Results from these simulations were then compared with the experimental results of the physical model. A percentage of deviation was calculated for the suspended solids deposited and the outflow of suspended solids. Results from some of these tests are shown in Table 13-1. An 18-inch pipe with a length of 795 feet was used in this comparison.

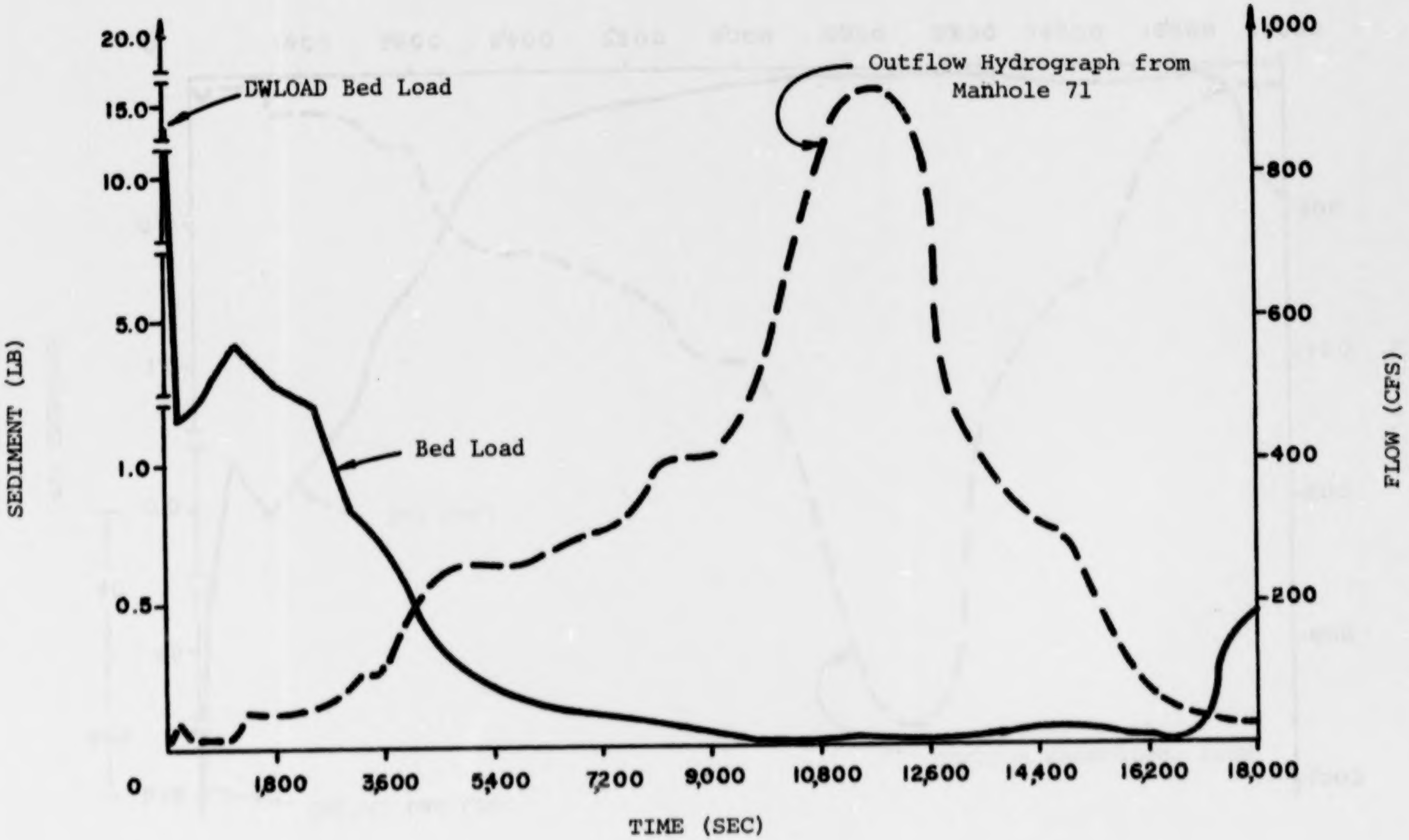

Figure 13-3. BED LOAD IN CONDUIT 72, STORM OF NOVEMBER 6, DWDAY=1

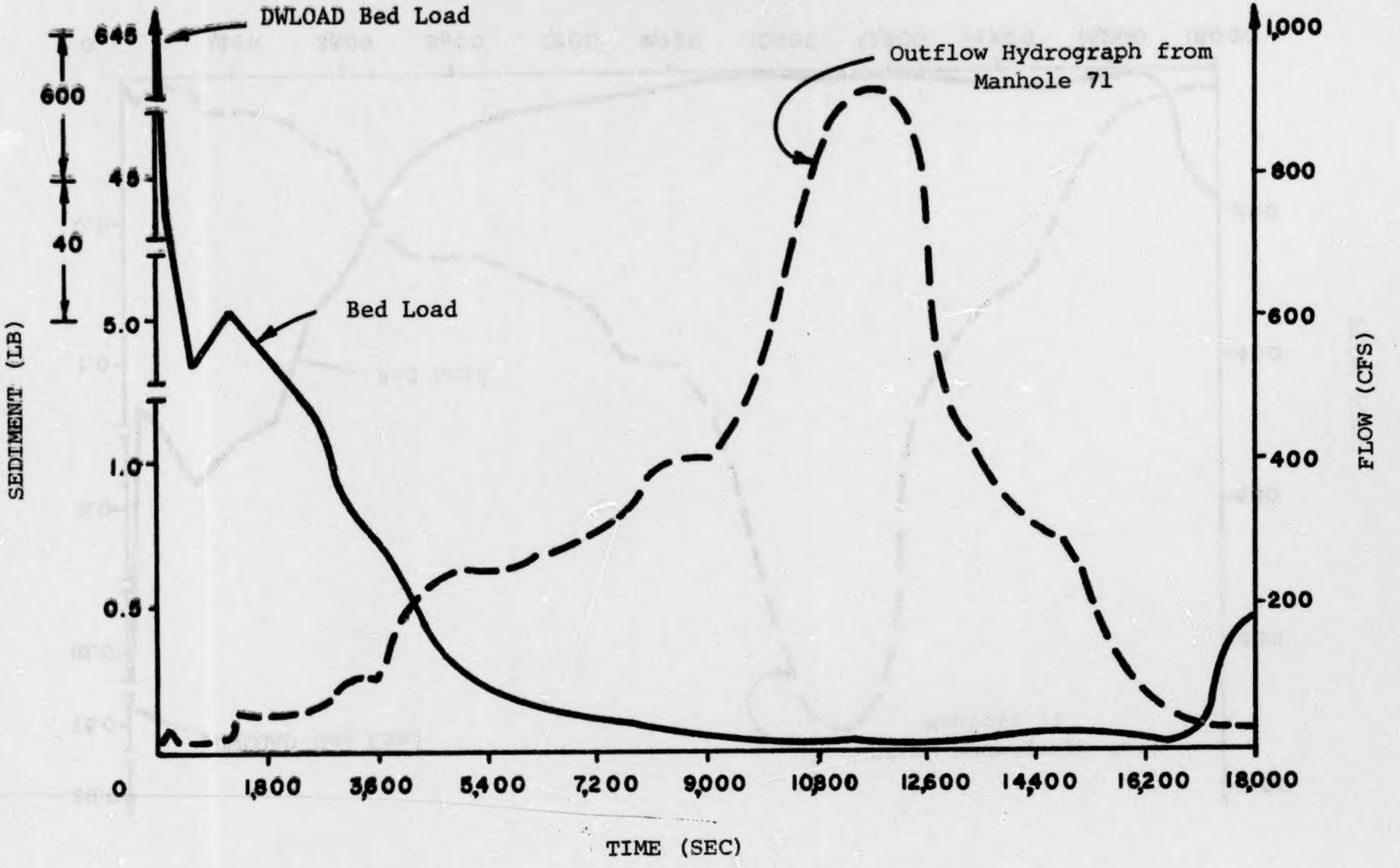

Figure 13-4. BED LOAD IN CONDUIT 72, STORM OF NOVEMBER 6, DWDAY=47

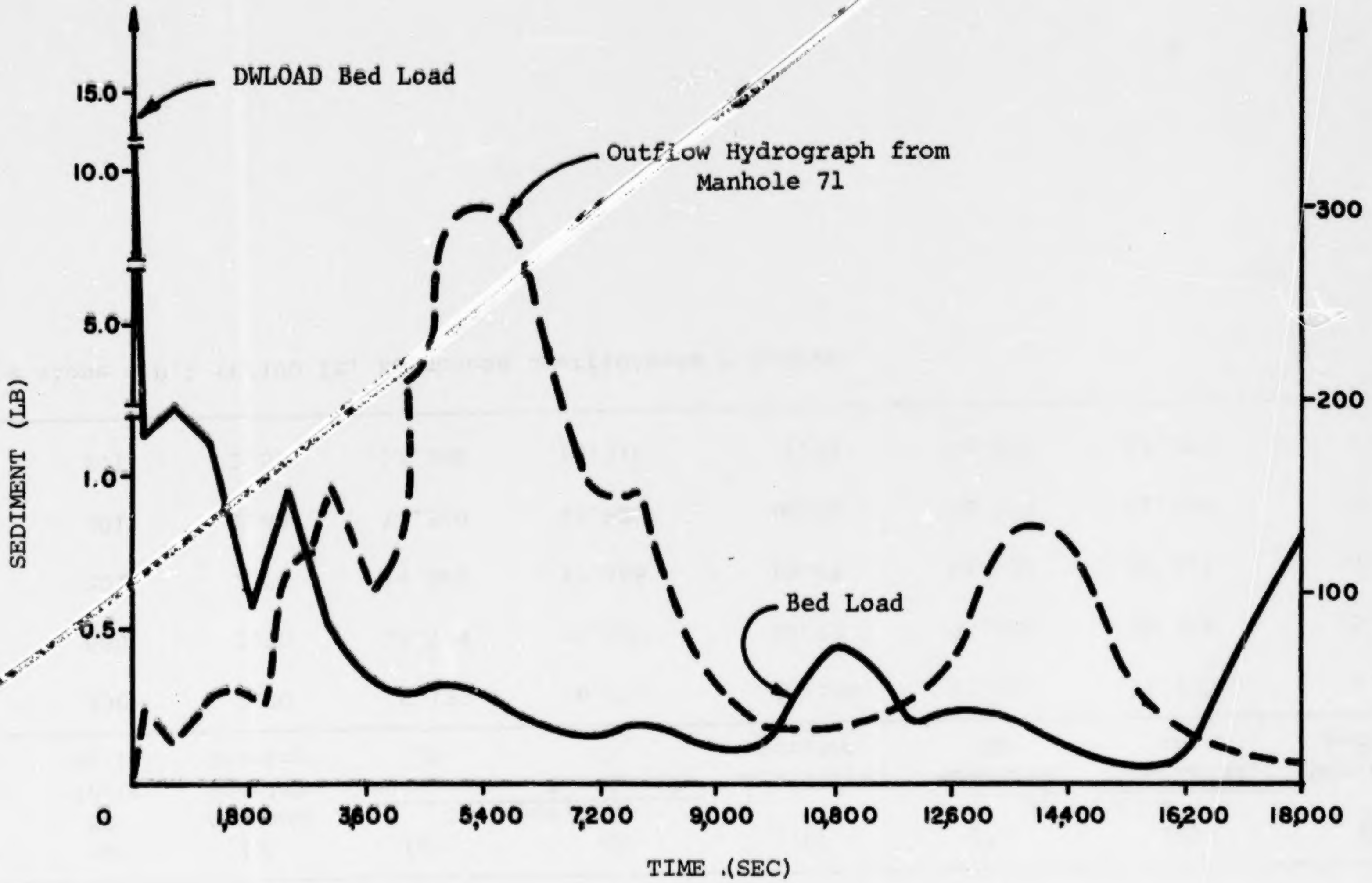

Figure 13-5. BED LOAD IN CONDUIT 72,
STORM OF NOVEMBER 14, DWDAY=1.0

Table 13-1. TEST RESULTS USING FMC PROJECT DATA, BED LOAD ANALYSIS FOR 18-INCH SEWER WITH A LENGTH OF 795 FEET*

| (1) | (2) | (3) | (4) | (5) | (6) | (7) | (8) | (9) |
|---|---|---|---|---|---|---|---|---|
|  |  |  | SS Deposited |  |  | SS Outflow |  |  |
| Flow, gpm | SS Input, mg/L | Assumed Specific Gravity | Measured, lb | Simulated, lb | Deviation, percent | Measured, lb | Simulated, lb | Deviation, percent |
| 50 | 230 | 2.70 | 5.133 | 8.573 | 40.13% | 57.167 | 51.977 | 9.98% |
| 10 | 686 | 2.70 | 15.744 | 23.233 | 32.23 | 35.656 | 28.356 | 25.74 |
| 30 | 201 | 2.70 | 14.369 | 17.165 | 19.45 | 61.131 | 58.415 | 4.44 |
| 30 | 201 | 2.49 | 14.369 | 12.923 | 10.06 | 61.131 | 62.659 | 2.50 |
| 30 | 201 | 2.51 | 14.369 | 13.319 | 7.30 | 61.131 | 62.263 | 1.82 |

*Pipe slope = 0.1 ft/100 ft; roughness coefficients = 0.0099.

Without further information on the characteristics of the suspended solids at the input, it is felt that practical assumptions are made concerning Shield's criterion, specific gravity, and the sewer sieve analysis relationship, as presented in Appendix B. Columns 4 and 5 in Table 13-1 illustrate a reduction in the percentage of deviation after a logical weighted average adjustment on specific gravity was made with the FMC data supplied on volatile solids.

## CONCLUSION

A general model has been developed which describes the movement of pollutants during transit in a prespecified sewer system. It has the capabilities of accounting for decay of nonconservative pollutants and allowing for sediment uptake and deposition when dealing with suspended solids.

The analyses performed verified the concepts used for complete mixing, continuity of mass, deoxygenation and reaeration, and sediment transport.

# SECTION 14

## RECEIVING WATER QUALITY MODEL

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# SECTION 14

# RECEIVING WATER QUALITY MODEL

## OBJECTIVES

The ultimate goal of the Storm Water Management program is to assess and evaluate the cost effectiveness of various pollution control measures. The objective of the Receiving Water Quality model is to provide a means for measuring the effect.

Runoff resulting from storm water carries with it various kinds of pollutants. Discharged into the receiving water, these pollutants are transported and dispersed into the aquatic environment. The job of the Receiving Water Quality model is to trace or route the pollutants, determining their temporal and spatial variation in the system.

Since the quality routing requires the receiving water quantity information, the model was developed to operate only in conjunction with the Receiving Water Quantity model described in Section 10. Both models use the same geometric representation which is capable of depicting the temporal and spatial distribution of pollutants as well as the hydrodynamics. To facilitate a rapid evaluation of the required removal by various treatment processes, however, the model is flexible in that it can perform several water quality simulations for a given quantity solution. This is desirable especially when many treatment processes modify the removal rate of pollutants rather than the change in hydraulic characteristics.

## RELATION TO STORM WATER PROGRAM

The Receiving Water Quality model receives storm water hydrograph and pollutograph inputs from either the Transport Block or the Storage Block through subroutine RECEIV of the Receiving Water Quantity model. It also accepts from RECEIV the quantity solution of the Receiving Water Quantity model.

Upon completion of the computations, the model prints and plots the concentration of various water quality constituents versus time for a selected number of junctions. The execution of the program will then be terminated.

## THEORETICAL DEVELOPMENT

Pollutants received from storm water and other inflows can basically be classified into three types, i.e., a conservative substance such as chloride ion; a nonconservative substance such as BOD; and a conservative substance, dissolved oxygen, that can be consumed by a nonconservative substance BOD. A conservative substance is defined as the one that does not decay itself, whereas a nonconservative substance is the one that will decay in proportion to the amount of material present, i.e., a first order reaction. There are other types of substances, but these three classes are of concern in the present model development.

### Quality Routing

Physical dispersion of a quality constituent in a receiving water is caused by transport phenomena, including advective transport and diffusion. If the quality constituent is nonconservative, it will be subjected to a continuous chemical reaction of decay. In addition, physical importation of water to the junction and/or extraction of water from the junction may carry with it the quality constituent. The algebraic sum of import and export may be termed a source or sink. The source and/or sink may include that of tidal exchange at the mouth of the estuary.

These mechanisms are operated simultaneously to modify the water quality constituent in all parts of the receiving water system, described by junctions. The basic equation describing these processes is the continuity equation. It can be written in finite difference form as follows:

$$\frac{\Delta c_j}{\Delta t} = - v \frac{\Delta c_j}{\Delta x} - kc_j + s_j \quad . \qquad (1)$$

where c = Constituent concentration

t = Time

v = Velocity averaged over the cross-section of the reach or channel

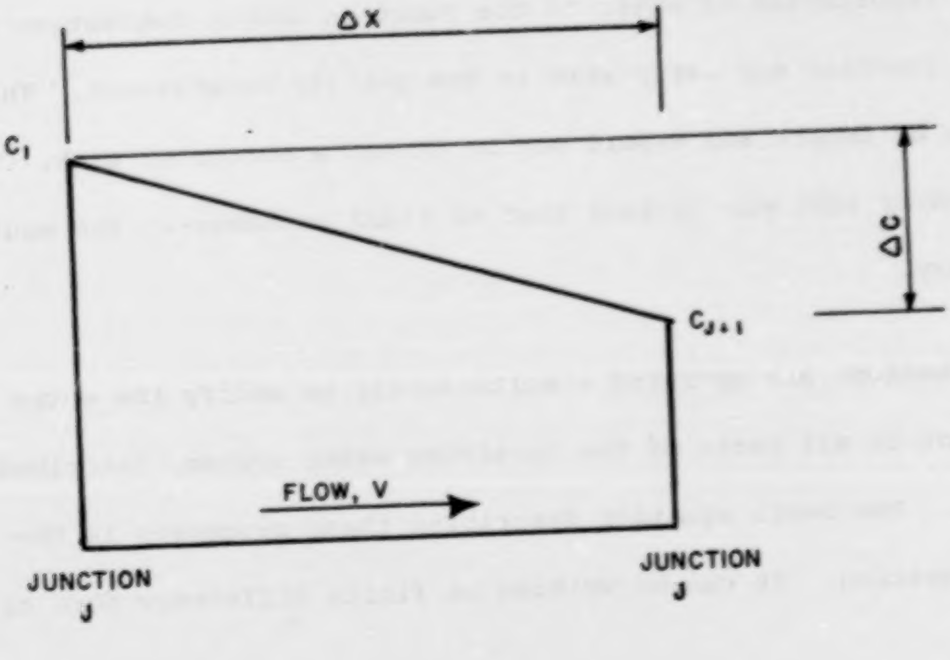
x = Distance

k = Decay constant

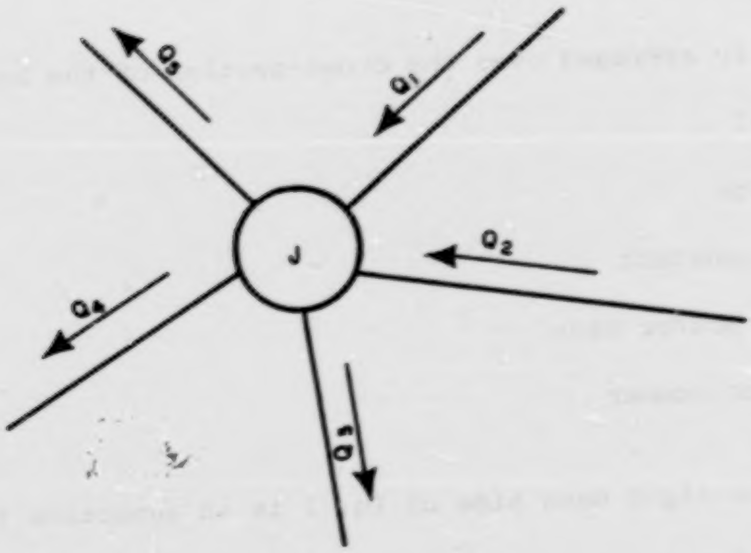
s = Source and/or sink

j = Junction number

The first term on the right hand side of Eq. 1 is an advective transport which, in the case of a non-branch system, is illustrated by Figure 14-1a. According to the expression, the advective transport simply translates


ΔX
$C_1$
ΔC
$C_{J+1}$
FLOW, V
JUNCTION
J
JUNCTION
J

(a) NON - BRANCH SYSTEM


$Q_5$
$Q_1$
J
$Q_2$
$Q_4$
$Q_3$

(b) BRANCH SYSTEM

Figure 14-1. ADVECTIVE TRANSPORT PHENOMENA

the linear concentration gradient along the flow path during the short period of time interval. During that period, the concentration at the downstream junction increases or decreases depending on the sign of concentration gradient.

It is noted that Eq. 1 does not include a term for diffusion transport. Experience has shown that, with the short computational time intervals used, the major transport mechanism is by advection and thus the diffusion term can be safely neglected.

For a branch system (Figure 14-1b) several flow paths may enter the junction. The concentration change due to advective transport is computed as a weighted average of transport due to all the channels. The flows in each channel are used as the weighting factors, thus

$$\frac{\Delta c_j}{\Delta t} = - \frac{\sum_{1}^{i} Q_i \, v_i \, \frac{\Delta c_{i,j}}{\Delta x_i}}{\sum_{1}^{i} Q_i} - K\,C_j + S_j \qquad (2)$$

where $Q$ = Flow volume

$i$ = Reaches entering or leaving the junction

Eq. 2 is the general expression of continuity for water quality routing. For a conservative substance, the decay coefficient will have a zero value.

Solution Procedure

Eq. 2 can be solved sequentially for all the junctions of the receiving water system. The velocity and flow in channels during each computational time-step are furnished by the Receiving Water Quantity model. The initial value of parameters is substituted into the equation to determine the changing rate of concentration. The concentration is continuously updated from one time-step to the next.

Sequential computation proceeds as follows:

1. Compute the rate change of concentration due to the advective transport.
2. Determine the amount of decay, if any.
3. Calculate the import and export amount.
4. Compute the new concentration of the quality constituent at the end of the period based on the above calculations.

These procedures are repeated for all the junctions before the computation is advanced to the next period. Six constituents can be solved simultaneously. When the solution for BOD is made, its exertion on DO and the resulting DO levels are determined simultaneously.

This is again the solution procedure for the familiar initial value problem. To increase the accuracy of computation, a modified Euler's method was used by instigating a half-time-step computation.

## Simulation Time Interval

Water quality routing does not have the severe stability problem found in the Receiving Water Quantity model solution. Experience has shown that an hourly interval of computation provides good results for both lake and estuary systems.

As a rule, the time-step for quality computation is much longer than that for quantity computation. The intermediate hydraulic results computed in each quality time-step (an hour) are therefore averaged for quality simulation.

### Linear Property

Because the continuity equation governing the water quality response as posed is linear, the pollutional effect of each point source can be computed separately. The overall effect is additive of each individual discharge provided that all the flow quantities of discharge have entered the quantity computation. This property simplifies the evaluation of the cost-effectiveness of the treatment by any given storm water polluter.

## THE MODEL SUBROUTINES

The program consists of four core subroutines, i.e., SWQUAL, INQUAL, LOOPQL, and QPRINT. SWQUAL is the subroutine called by RECEIV to take over control of the quality subroutine with the assistance of other subroutines.

INQUAL is the subroutine for reading input data. Depending on the number of days to be simulated, SWQUAL will call LOOPQL once per day to perform the quality routing described previously. QPRINT prints quality response for various constituents at a selected number of time-steps and junctions for engineering interpretation of results.

## TEST APPLICATION

The general method of quality routing described herein has been proven valid for several prototype systems and conditions receiving waste load inputs other than the impulse type pollutograph (Refs. 1,2,3). Actual data, however, were not available for the verification of the receiving water quality dynamic response to the storm water pollution.

Hypothetical cases were therefore used to test for the functionality of the model. This was done by imposing a wide variety of input conditions on an assumed receiving water system. The system response was then evaluated and compared with the system behavior that can reasonably be expected to occur.

The hydraulic system used for the quality routing is the same as the one used for the Receiving Water Quantity Model (see Figure 10-1).

Briefly, each channel was 10,000 feet long, with a depth ranging from 15 feet at channel 28 to 20 feet throughout the open water body. Dry weather flow was 2,000 cfs at junction 18, 500 cfs at junction 16, 50 cfs at junctions 10 and 14, and 20 cfs at junction 13. Input hydrographs were superimposed on the dry weather flow and a prescribed sinusoidal tide was imposed on junction 1.

In addition to hydraulic inputs, the Receiving Water Quality model accepted a waste input to junction 14 as shown in Figure 14-2. A similar waste input with a peak value of 250,000 pounds per day was imposed on junction 13. The model was asked to evaluate the quality impact of waste discharge from these particular point sources. Note

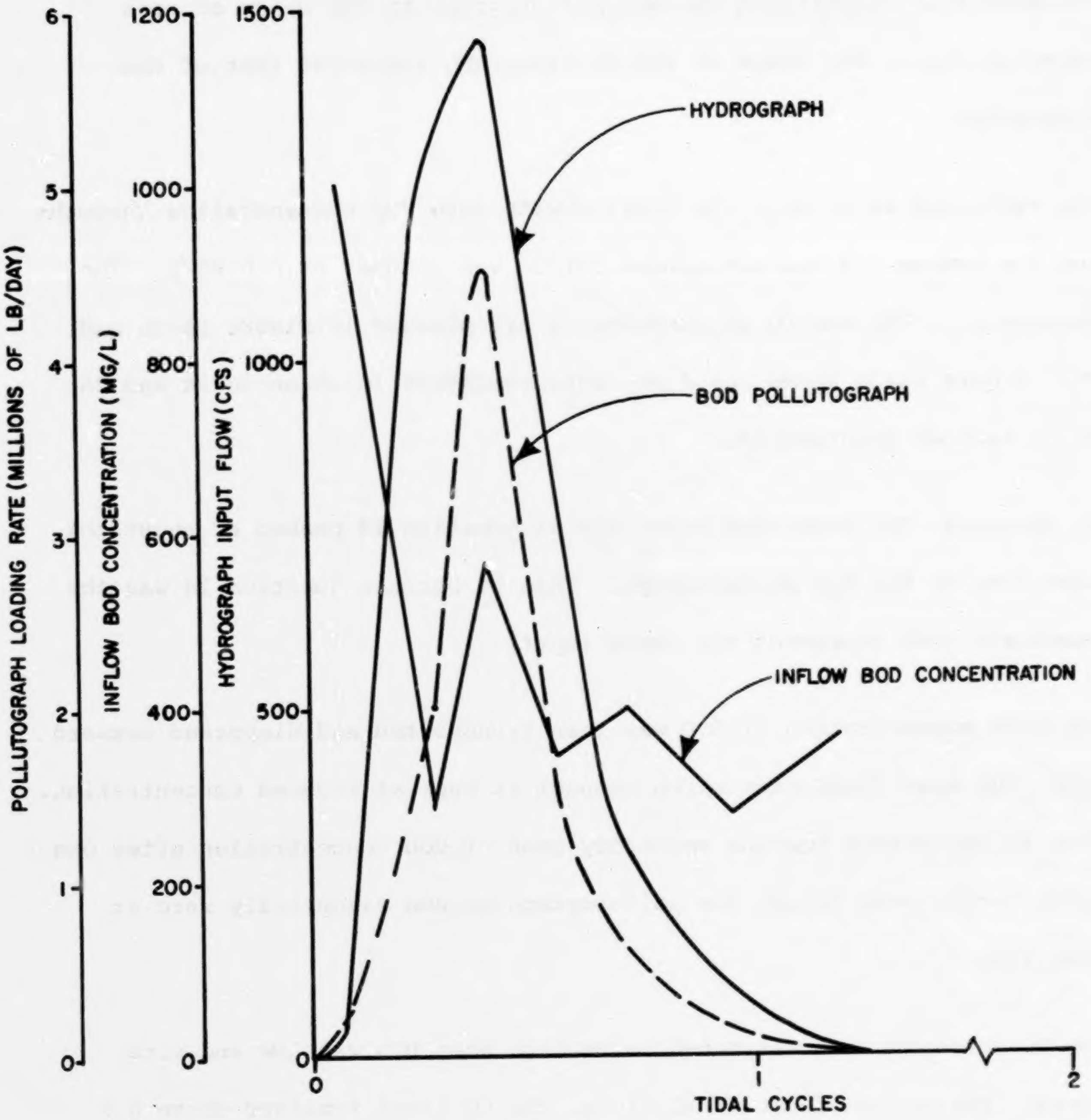


Figure 14-2. INPUT TO NODE 14 OF TEST SYSTEM

that the inflow BOD concentration decreased with time as it is typically observed in the field. The product of the hydrograph and the inflow concentration constituted the BOD pollutograph in the units of mass emission rate. The shape of the pollutograph resembled that of the hydrograph.

The receiving water body was started with zero BOD concentration throughout the system and the saturation for DO was assumed as 8.0 mg/L. The responses of BOD and DO at junction 14 are plotted in Figure 14-3b and 3c. Figure 14-3a shows the flow characteristics in channels 24 and 25 which meet at junction 14.

As expected, the receiving water BOD at junction 14 peaked at about the same time as the BOD pollutograph. This is because junction 14 was the immediate node receiving the waste input.

The high concentration of BOD was then transported and dispersed seaward until the next flood tide which brought it back at reduced concentration. This is the reason for the secondary peak of BOD concentration after one tidal cycle, even though the pollutograph became essentially zero at that time.

The DO concentration was noted to be high when BOD was low and vice versa. During the first tidal cycle, the DO level remained above 6.6 mg/L. This is understandable because the water volume flowing through junction 14 was relatively high and thus it provided a large resource of DO for BOD exertion. During the second tidal cycle, the DO level

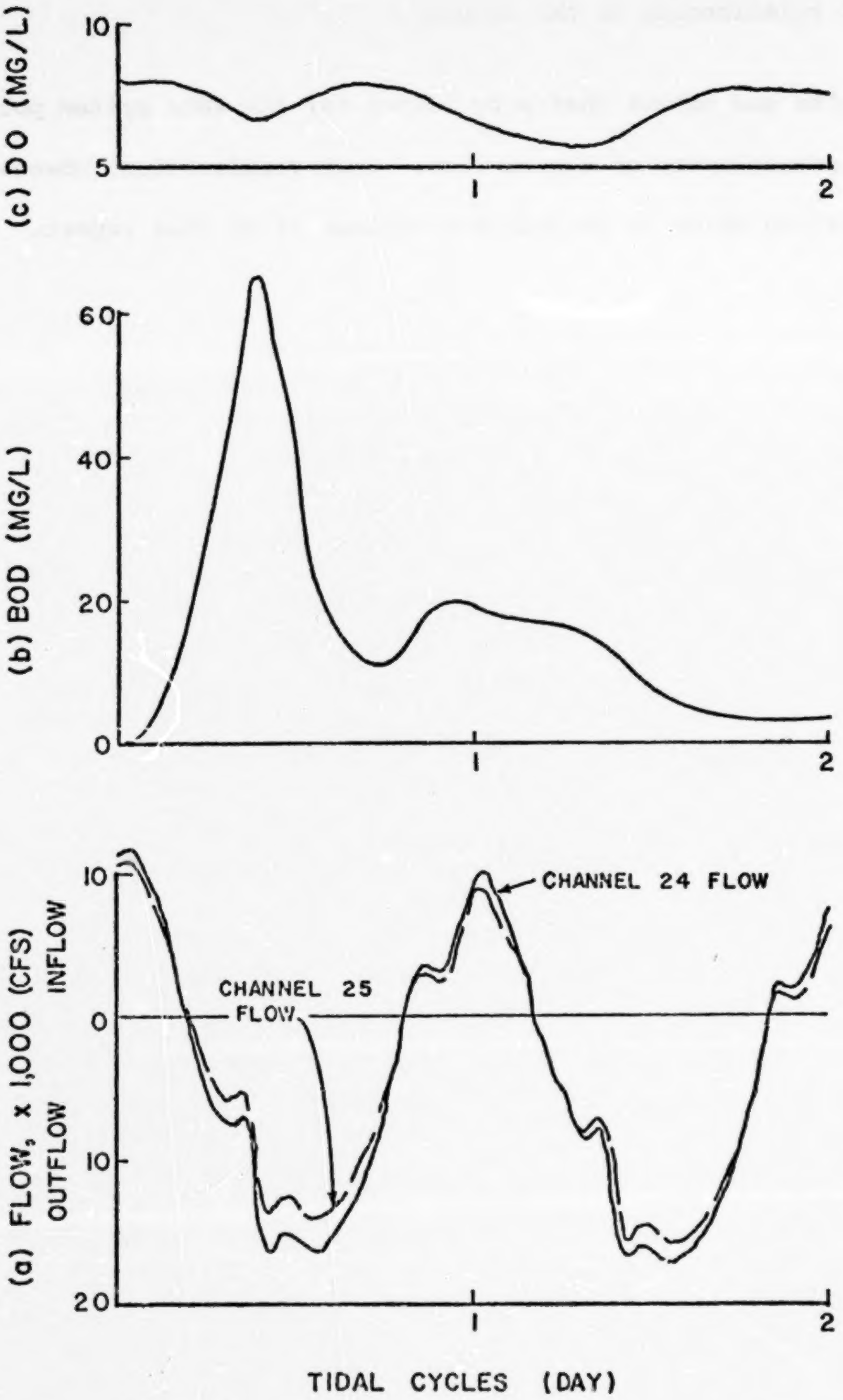


Figure 14-3. RECEIVING WATER QUALITY MODEL RESULTS FOR NODE 14 OF TEST SYSTEM

was lowered to 5.3 mg/L, resulting from the accumulative effect of the BOD and DO relationship in the estuary.

These results and others that were tested for the lake system pointed out the reasonableness of the answers. Real verification, however, awaited testing which is described in Volume II of this report.

## SECTION 15

## TREATMENT MODEL

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## SECTION 15

## TREATMENT MODEL

### OBJECTIVES

The objectives of the Treatment model are to:

1. Provide the capability of modeling a considerable number of alternative sewage treatment processes to be located at or near a sewer outfall, and to restrict certain process combinations which are deemed inadmissible.
2. Simulate improvements in overflow quality produced by each component of the selected combination of treatment processes.
3. Summarize data required for the estimation of treatment costs for the specified installation.

### BACKGROUND

Processes applicable to the treatment of overflows from combined sewers and storm sewer discharges should meet the following criteria:

1. They should operate efficiently when called upon after varying and extended periods of inactivity.
2. They should have large throughput capacity.
3. They should be simple and compact, requiring as little land area as possible.
4. They should not produce odors, either during or between storms.
5. The benefit to the community should be worth the cost.

To meet these criteria, there are available the existing primary treatment processes, supplemented by chlorination, which are familiar to all sanitary engineers, plus a variety of ultrafine straining and filtration processes now undergoing intensive investigation under FWQA sponsorship. Data incorporated in the model relative to these latter processes are based on currently available information and may require revision when results of the completed investigations are published.

Storage, either alone or combined with sedimentation, may be used to reduce the hydraulic capacity of the treatment units and is incorporated in the model.

The first criterion eliminates the use of biological processes, except in specially favorable situations, because of the difficulty and cost of maintaining a large supply of active biological media during dry weather to have available in sufficient quantity during storms. For this reason biological processes are not included in the model.

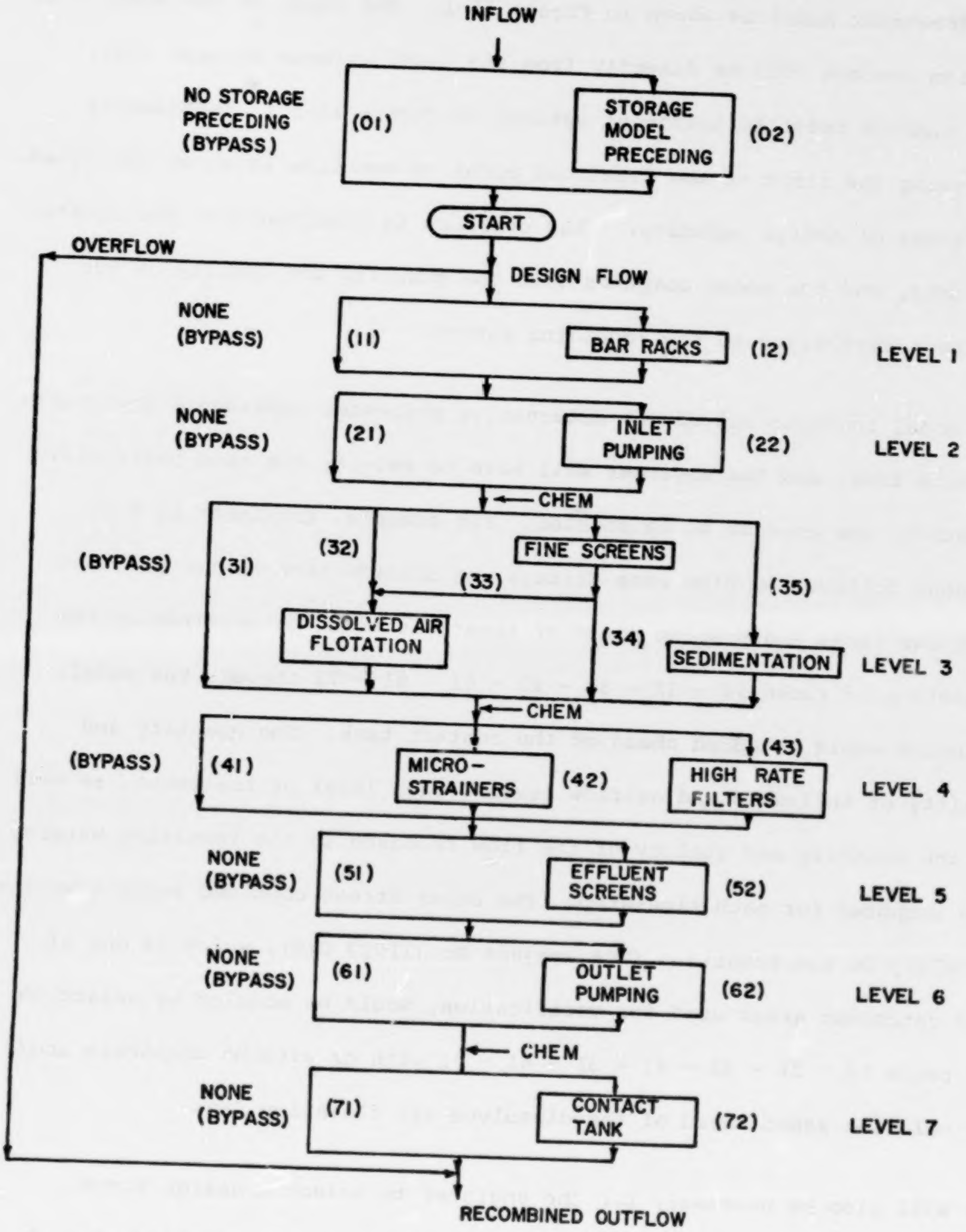
## THE MODEL SUBROUTINE

The Transport Model will provide a hydrograph of flow and pollutographs of BOD, suspended solids, and coliforms at the point of discharge. These either will pass directly to the Treatment model or will be routed through the Storage Model first. In either case the input to the Treatment model will consist of a hydrograph of flow and pollutographs of BOD, suspended solids, and total coliforms (most probable number) for each storm event analyzed. Additional pollutographs for other polluting constituents may be added in the future when data become available.

The Treatment model is shown in Figure 15-1. The input to the model will be from storage (02) or directly from the sewer without storage (01). (The numbers refer to treatment options in Figure 15-1.) Immediately following the start of the Treatment model an overflow is shown for flows in excess of design capacity. This overflow is combined with the treated effluent, and the model computes both the quantity and quality of the effluent discharged to the receiving waters.

The model includes all of the alternative processes considered applicable at this time, and the engineer will have to select, for each particular location, the process to be modeled. For example, treatment by fine screens followed by high rate filters and disinfection of the effluent with bar racks and pumping ahead of treatment would be obtained by the selection of route 12 - 22 - 34 - 43 - 51 - 61 - 72 through the model. Chlorine would be added ahead of the contact tank. The quantity and quality of inflow to and outflow leaving each level of treatment, as well as the quantity and quality of the flow released to the receiving waters, are computed for each time-step. The Baker Street combined sewer overflow facility in San Francisco (EPA Project No. 11023 DXC), which is one of the catchment areas used for verification, would be modeled by selection of route 12 - 21 - 32 - 41 - 51 - 61 - 71 with or without chemicals and/or chlorine added ahead of the dissolved air flotation tank.

It will also be necessary for the engineer to select a design storm event so that the model can select the number and size of units of each type required by the process selected.


INFLOW
NO STORAGE PRECEDING (BYPASS)
(01)
STORAGE MODEL PRECEDING
(02)
START
OVERFLOW
DESIGN FLOW
NONE (BYPASS)
(11)
BAR RACKS
(12)
LEVEL 1
NONE (BYPASS)
(21)
INLET PUMPING
(22)
LEVEL 2
CHEM
(BYPASS)
(31)
(32)
FINE SCREENS
(33)
(35)
DISSOLVED AIR FLOTATION
(34)
SEDIMENTATION
LEVEL 3
CHEM
(43)
(BYPASS)
(41)
MICRO-STRAINERS
(42)
HIGH RATE FILTERS
LEVEL 4
NONE (BYPASS)
(51)
EFFLUENT SCREENS
(52)
LEVEL 5
NONE (BYPASS)
(61)
OUTLET PUMPING
(62)
LEVEL 6
CHEM
NONE (BYPASS)
(71)
CONTACT TANK
(72)
LEVEL 7
RECOMBINED OUTFLOW

NOTE: Numbers in parentheses label treatment options.

Figure 15-1. TREATMENT MODEL

The design of the plant is based on combinations of modules of specific sizes, resulting in plants having the following maximum hydraulic capacities in mgd: 5, 10, 15, 20, 25, 30, 35, 50, 75, 100, 125, 150, 200, 250, 300, 350, 400, 450, and 500. The engineer will select the percentage of the peak flow of the design event for which he wishes to size the units (80 percent is suggested), and the model will automatically size the plant for the next larger modular flow from the above list. Alternatively, the engineer may select a design flow irrespective of the storm to be handled, and the model will design the plant for the modular flow equal to or (next) greater than the selected flow. After the model has computed the number and size of units, it can be used to compute the operating performance for any storm or succession of storms in a given study period.

The model will sum up the flow treated, the BOD, suspended solids, and coliforms in the influent and effluent of the complete combination of treatment processes, and the removals in each level of treatment. It will compute the treatment efficiency with and without allowance for overflow quantities bypassing the treatment units. It will also compute the construction cost, including the cost or value of the land required and the annual cost of operation and maintenance.

## PUMPING STATIONS

The need for pumping stations will depend on the topography and the available head. It is assumed that sufficient variable speed pumps are included in each pumping station so that the rate of pumping will exactly equal the inflow rate. The inclusion or omission of a pumping station will therefore have no effect on the hydrograph and pollutographs of the flows to be treated. The size of the pumping station necessary to handle the design event, the head, and the quantity pumped must be computed so that the costs can be determined.

## THEORETICAL DEVELOPMENT (Treatment Processes)

The following discussion of the various treatment processes will include the criteria adopted for selecting the number and size of units, and the operating efficiency adopted for each process, together with the basis therefore. (For the actual working of each model subroutine see the "User's Manual." For the cost figures incorporated in the model, see Section 16.)

### Bar Racks (Treatment Option 12)

It was assumed that bar racks would be required in all treatment systems, except where sedimentation occurring in storage units is the only or primary treatment. The Treatment model handles the latter process by selection of route 02 - 11 - 21 - 35 which may be followed by filtration or microstraining and disinfection.

It was assumed that bars would be spaced from 1-inch clear opening to 2-inches on centers, and that the racks would be mechanically cleaned with operation initiated by a float switch. There will be at least two units in each installation, so that if one unit is undergoing maintenance, at least half of the flow capacity and probably somewhat more will be available at all times. Units will be sized for a 3.0-fps velocity in the approach channel at peak design flow and will be installed at a slope of 1 horizontal on 2 vertical. Channels will be designed for 2-feet freeboard. It was assumed that the maximum size of units would be 8 feet submerged depth by 10 feet wide, or a vertically projected submerged area of 80 square feet. This results in a maximum capacity per bar rack of 240 cfs.

The quantity of screenings removed is based on screenings removal at municipal sewage treatment plants. Screenings removal varies from 0.1 to 5 cubic feet per million gallons (Ref. 1), but very few cities report more than 2.5 cubic feet per million gallons. For purposes of this study, 2.0 cubic feet per million gallons has been taken as a liberal average. These are annual average results, and it is a well known fact that the quantity of screenings increases greatly during storms, especially for combined sewer systems. It is not known whether the proportion of screenings in the overflows will be as great as in the intercepted flows, but to be on the safe side it has been assumed that they will. A factor of 3 has been used to allow for the greater quantity of screenings in storm flows, or 6 cubic feet per million gallons.

If it is assumed that screenings weigh 50 pounds per cubic foot and contain 85 percent moisture, the dry solids removal amounts to 5.4 mg/L. Solids of this type are generally excluded from samples subjected to suspended solids analysis. Nevertheless, in order to indicate some treatment effectiveness, 5.4 mg/L has been used for suspended solids removal and 5 percent of that figure has been used for BOD removal.

Fine Screens (Treatment Options 33 and 34)

Fine screens, popular as an alternate to sedimentation tanks 50 years ago and still in use in some locations such as Milwaukee, have slotted holes varying from 3/64-inch to 1/8-inch wide by 2 inches long. A number of manufacturers have been making rotary drum screens for industrial processing, including the screening of industrial waste waters before discharge. These screens are similar to the well publicized microstrainer but with a coarser wire mesh. The screen wire normally available varies from 6-mesh to 60-mesh. The openings in the 60-mesh wire are .009 inch, equivalent to 230 microns. It is anticipated that fine screens of this type could provide economical treatment of storm water overflows. A pilot plant (EPA Project No. 11023 FDC) containing such a screen with 50-mesh wire (openings equal to 297 microns) has been operated on storm overflows in Milwaukee in combination with a dissolved air flotation unit (Ref. 2) and has provided significant data for both processes, individually and in combination. The modeling of the fine screens is based almost entirely on the data in Ref. 2.

The pilot plant (Ref. 2) was operated at a hydraulic loading rate of 50 gpm per square foot and this rate has been adopted for sizing the units. At least two units would be installed, with the total capacity of all units equalling the design flow for the modular plant.

The removal of suspended solids by the screen in Ref. 2 varied from 24.9 ± 9.8 percent during the summer and fall to 28.8 ± 10.5 percent in the spring. The removal of BOD varied from 20.3 ± 6.5 percent in the summer and fall to 23.4 ± 9.3 percent in the spring. Sufficient data are not available to correlate removal efficiency with storm water quality. Removal efficiency will undoubtedly vary with the size distribution of solids, and hence possibly with the time of flow in the sewers. For the present, the Treatment model assumes an average removal of 27 percent of the suspended solids and 22 percent of the BOD. The backwash rate has been assumed to be 0.75 percent of the flow. This flow will be pumped or will flow by gravity to a sanitary sewer or interceptor for transportation to the sewage treatment plant.

### Sedimentation (Treatment Option 35)

Sedimentation in sedimentation tanks or storage units is an obvious way to improve the quality of sewer overflows. The sizing of sedimentation tanks and their treatment efficiency depends primarily on the overflow rate. Similarly, the treatment efficiency due to sedimentation in storage units will depend primarily on the overflow rate, but the overflow rate in this case will be based on the design of the basin as a storage unit and not as a sedimentation tank. Types of flow and pollutant increments through larger storage units are discussed in Section 9.

Primary sedimentation tanks in municipal sewage treatment plants were formerly designed for detention periods of one to two hours, but if the depth is fixed, the overflow rate is also fixed. Regulatory agencies now limit overflow rates to 600 gpsf per day for small plants (less than 1.0 mgd), but permit up to 1,000 gpsf per day for larger plants where the tanks are followed by secondary treatment units. Since the maximum flow at sewage treatment plants normally varies from 1.5 to 2.0 times the average design flow and may be as high as 2.5 to 3.0 times the average design flow, especially at small plants, it is logical to increase the overflow rate for tanks designed on a peak flow basis. The Treatment model provides for selection of the overflow rate by the engineer. An overflow rate of 800 times 2, or 1,600 gpsf per day is suggested. An average tank water depth of 8 feet is assumed.

The removal of suspended solids decreases as overflow rates increase (Ref. 1, p. 93) and as detention periods decrease (Ref. 3, p. 338, and Ref. 4, p. 610). It is also evident from Ref. 3 that the removal efficiency increases as the strength of the sewage increases.

The data in Tables 9. 10, and 11 of Ref. 1 were plotted with percent removal of suspended solids as ordinates versus overflow rates as abscissas. Figure 15-2 is a plot of only those tanks with influent suspended solids concentration between 100 and 200 ppm. The average removal is given by the following equation:

$$\text{Removal} = .70 - .40\left(\frac{\text{OVFRA} - 300}{2{,}000}\right) \qquad (1)$$

where OVFRA = Overflow rate in gpm/sq ft/day

If the percentage removals for more concentrated sewages are plotted on similar diagrams, the average removal curves are displaced upward parallel to the line on Figure 15-2. The additional removal amounts to:

$$\left(\frac{\text{SS conc.} - 140}{190}\right) \times .06 \text{ or } \frac{\text{SS conc.} \times .06}{190} - .044 \qquad (2)$$

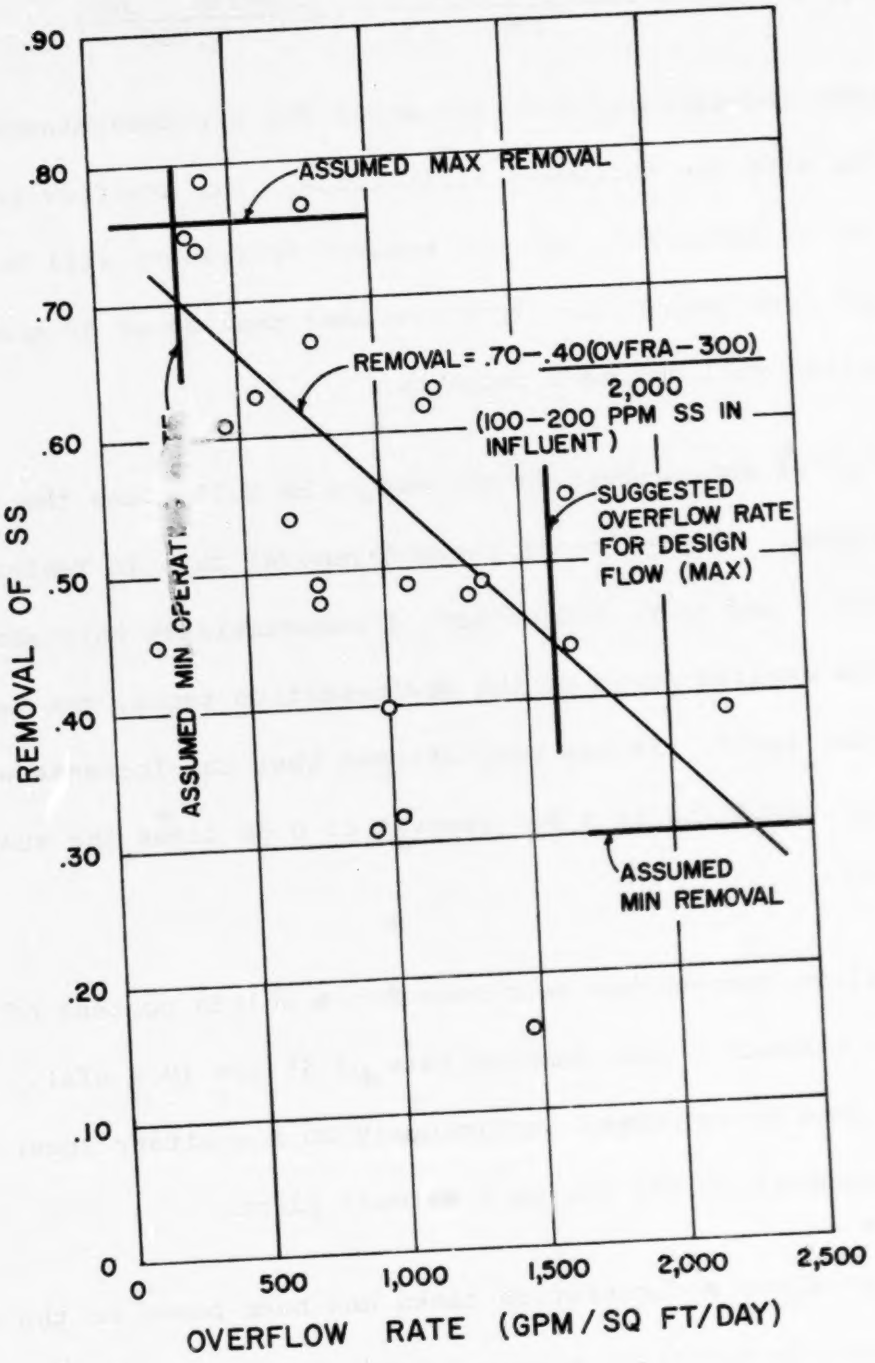


Figure 15-2. TREATMENT OF OVERFLOWS BY SEDIMENTATION TANKS

When Eq. 2 is combined with Eq. 1 the formula for removal of suspended solids becomes:

$$\text{Removal} = 0.656 + \frac{\text{SS conc. x .06}}{190} - .40\left(\frac{\text{OVFRA} - 300}{2{,}000}\right) \qquad (3)$$

Eq. 3 has been incorporated into the model for the determination of suspended solids with the following limitations: the overflow rate will be assumed to be at least 300, and the removal efficiency will be not less than 0.30 nor more than 0.76. It is assumed that scour of already deposited solids will not be a problem.

The percentage of BOD removed is assumed to be 0.55 times the removal of suspended solids. Inspection of the BOD removal data in Tables 9, 10, and 11 of Ref. 1 and on p. 610 of Ref. 4 substantiates this assumption. If chlorine is applied ahead of the sedimentation tanks, the removal of BOD will be increased. It has been assumed that the increase will amount to 20 percent, resulting in a BOD removal of 0.66 times the suspended solids removal.

The sludge volume removed has been based on a solids content of 5 percent, and also on a minimum sludge pumping rate of 45 gpm (0.1 cfs). The sludge is assumed to be pumped continuously to a sanitary sewer or interceptor for transport to the sewage treatment plant.

The efficiency of the sedimentation tanks has been based on the efficiency of similar tanks in municipal sewage treatment practice. Somewhat different results may be obtained from sedimentation tanks installed at the outlet of storm drains, especially where soil erosion is a factor.

For combined sewers where soil erosion is not a factor, the data used appear to be applicable without further adjustment.

## Dissolved Air Flotation (Treatment Option 32)

A dissolved air flotation plant is under construction on the Baker Street outfall into San Francisco Bay. When operating data from this facility become available, the following determination of the treatment efficiency of dissolved air flotation units should be reviewed and revised if necessary. The plans and specifications as well as the engineers' preliminary reports for this project were made available for this study. The facility is designed to handle 15 percent of the peak flow from a five-year storm, or a runoff rate of 0.20 inches per hour at an overflow rate, not including a recycle flow of 4,050 gpsf per day. The flotation tanks will be preceded by mechanically cleaned bar racks, and excess flows will be bypassed. Facilities will be installed for the addition of chemical coagulants and chlorine.

The data in Ref. 2 for the combined operation of fine screens and dissolved air flotation has also been useful. Based on Ref. 2, the fine screens removed 27 percent of the suspended solids and the dissolved air flotation process removed 33 percent of what was left, for an overall removal of 51 percent. Since the dissolved air flotation tank would contain sludge removal equipment as well as scum removal equipment, it was assumed that the same percentage of removal would be obtained without the fine screens at the cost of somewhat more air and possibly a slightly greater recycle rate. This efficiency was attained in the pilot plant at an overflow rate of 3 gpm per square foot (4,320 gpsf per day) with no recycle.

(A portion of the effluent from the fine screens was pressurized instead of recycled flotation tank effluent. Recycling may be more practical in a full size plant.) No tests were made at other overflow rates but additional tests at higher rates of flow are contemplated.

The model allows the engineer to select the overflow rate based on design flow plus recirculation, the percent of recycle flow, and the tank depth. An overflow rate of 5,000 gpsf per day, 15 percent recycle, and a tank depth of 10 feet are suggested for normal design.

It was assumed that the removal efficiency would be comparable to the removal efficiency of plain sedimentation tanks, but that this efficiency would be obtained at higher overflow rates due to the rapid rise of the air bubbles. Since the 51 percent removal reported for the pilot plant of Ref. 2 at an overflow rate of 4,320 gpsf per day is obtained (in plain sedimentation tanks treating 100-200 ppm suspended solids at an overflow rate of 1,250 gpsf per day), we have used the percentage removals for plain sedimentation, but for overflow rates equal to 4,320/1,250, or 3.5 times the rates for plain sedimentation. This leads to the following equation for the removal of suspended solids:

$$\text{Removal} = 0.656 + \frac{\text{SS conc.} \times .06}{190} - .40\left(\frac{\text{OVFRA} - 1,000}{7,000}\right) \qquad (4)$$

In using Eq. 4 and the following modifications a minimum overflow rate of 1,000 gpsf per day is assumed by the model.

The addition of chemicals increased the overall removal of suspended solids from 51 percent to 68.3 percent (Ref. 2). The actual removal can be expected to decrease as the overflow rate increases. To allow for this factor, the increased removal due to chemicals was assumed to be

$$0.20 - \left(\frac{\text{OVFRA} \times .10}{10{,}000}\right) \quad (5)$$

or 15 percent at an overflow rate of 5,000 gpsf per day. This can be simplified to

$$\left(\frac{20{,}000 - \text{OVFRA}}{100{,}000}\right) \quad (6)$$

and this term has been added to the removal equation when chemicals are added. The complete equation becomes

$$\text{SSREMV} = .656 + \frac{.06 \times \text{SSINF}}{190} - .40\left(\frac{\text{OVFRA} - 1{,}000}{7{,}000}\right) + \text{ICHEM}\left(\frac{20{,}000 - \text{OVFRA}}{100{,}000}\right) \quad (7)$$

where SSREMV = Suspended solids removed

SSINF = Suspended solids in inflow

ICHEM = 0 if no chemicals are added

ICHEM = 1 if chemicals are added

The amount of chemicals added is computed at 12 mg/L, based on Ref. 2. Regardless of the equations the model will restrict the removal of suspended solids to not less than 0.20 and not more than 0.82.

The reported results in Ref. 2 indicate a BOD removal without the addition of chemicals of approximately 90 percent of the suspended solids removal. If chemicals are added the increase is only 2 percent. The chemicals would have little effect on the soluble BOD. If chlorine is added about 2 ppm of BOD is eliminated per 1 ppm of chlorine used up.

It is assumed that this will amount to an average increase in BOD removal of 15 percent. Adopting the above as the basis for BOD removal, the following equation is obtained:

$$\text{BODREM} = .59 + \frac{.05 \times \text{BOD conc.}}{100} - .36\left(\frac{\text{OVFRA} - 1{,}000}{7{,}000}\right) + \text{ICHEM} \times .02 + \text{ICL2} \times .15 \tag{8}$$

where ICHEM and ICL2 equal 1 or 0, depending on whether or not chemicals and/or chlorine are added. In order to keep the percentage removals of BOD within practical limits for all contingencies, the model restricts the BOD removal (BODREM) to not less than 0.18 and not more than 0.60.

The amount of chlorine used is taken at 10 mg/L if the BOD of the influent is 130 or less, and at 15 mg/L if it is more.

The volume of sludge and scum is taken equal to 1.5 percent of the flow (Ref. 2). This will be pumped to a sanitary sewer or interceptor for transport to the sewage treatment plant.

The detention time in the flotation tank will be computed and if it is less than 15 minutes it will be used in subroutine KILL to compute the removal of coliforms. (See Chlorination discussion.)

## Dissolved Air Flotation Preceded by Fine Screens (Treatment Option 33)

For the combination of fine screens followed by dissolved air flotation it has been assumed that the fine screens would perform exactly as described earlier, i.e., they would remove 27 percent of the suspended solids and 22 percent of the BOD, regardless of the flow rate or concentration of pollutants.

Since tests of this combination of processes in Ref. 2 were used to establish the performance of dissolved air flotation alone at an overflow rate of 4,320 gpsf per day, it follows that the overall efficiency of removal predicted by the model should be the same in treatment option 33 with fine screens as in treatment option 32 without fine screens at an overflow rate of 4,320 gpsf per day. However, since part of the suspended solids and BOD has been removed by the fine screens, a different removal efficiency must be applied to the fine screen effluent. No test results of the combination at other overflow rates are available, but it is reasonable to assume that the overall results of the combination would vary with the overflow rate in about the same manner as for dissolved air flotation alone. Accordingly, the equations for suspended solids and BOD removal for dissolved air flotation alone were adjusted to give approximately the same overall removals at overflow rates of 1,000 and 4,320 gpsf per day, when combined with the removals by the fine screens. To obtain comparable removal efficiencies it was also necessary to limit the minimum overflow rate to 1,000 gpsf per day as before, and to impose reasonable limits on the maximum and minimum values of suspended solids and BOD removal whether or not chemicals and/or chlorine are added.

The resulting equations, to be applied to the effluent from the fine screens, are as follows:

$$\begin{aligned} \text{SSREMV} = {} & 0.528 + \frac{\text{SS conc. x .06}}{190} - 0.486\left(\frac{\text{OVFRA} - 1{,}000}{7{,}000}\right) + \\ & \text{ICHEM x } 1.37\left(\frac{20{,}000 - \text{OVFRA}}{100{,}000}\right) \end{aligned} \tag{9}$$

$$\begin{aligned} \text{BODREM} = {} & 0.475 + \frac{\text{.05 x BOD conc.}}{100} - 0.405\left(\frac{\text{OVFRA} - 1{,}000}{7{,}000}\right) + \\ & \text{ICHEM x } 1.30 \text{ x } 0.02 + \text{ICL2 x } 1.30 \text{ x } 0.15 \end{aligned} \tag{10}$$

where ICHEM = 1 if chemicals are applied and 0 if chemicals are not applied

ICL2 = 1 if chlorine is applied and 0 if chlorine is not applied

Regardless of the above equations, the model will limit the suspended solids removal to not more than 75 percent and not less than 15 percent, and the BOD removal to not more than 48 percent and not less than 15 percent.

In comparing the performance of dissolved air flotation tanks with and without fine screens ahead of them, different overflow rates and recirculation rates may be assumed for the two cases.

Microstrainers (Treatment Option 42)

A microstrainer consists of a rotating drum covered with screen cloth operating partly submerged in a concrete tank. The water enters one end of the drum and flows outward through the screen cloth. As the drum revolves the solids caught on the inside of the cloth are carried to the top of the drum where they are washed into a trough by high pressure jets of strained effluent and discharged to waste. Microstrainer screen cloth comes in three mesh sizes, designated Mark 0, I, and II with effective aperture sizes of 23, 35, and 60 microns. Developed originally in England for straining algae out of water supplies, microstrainers have been used in England for polishing sewage treatment plant effluents since 1950 (Ref. 5), and have recently been undergoing tests in this country to determine their suitability as tertiary treatment units on activated sludge effluents at Lebanon, Ohio (Ref. 6), and Chicago (Ref. 7).

Under a contract with the EPA a microstrainer with a diameter of 5 feet and a length of 3 feet was installed on the overflow of a combined sewer serving the 11.2-acre Callowhill residential area in Philadelphia. Preliminary data are contained in Ref. 8. This filter had a submerged area of about 30 square feet, and the individual capacities of the pumps supplying the filter were 5,000 and 12,000 gph for a combined capacity of 17,000 gph or 284 gpm, roughly equivalent to 9.5 gpm per square foot of submerged area. Initial tests were made with Mark I cloth, but Mark 0 cloth was substituted after six months resulting in greatly improved efficiencies. No difficulties due to plugging of the screen were experienced with either cloth.

At Lebanon, a microstrainer with a diameter of 5 feet and a length of 1 foot was operated on activated sludge effluent for a period of five months. Capacity at a 6-inch head loss was 3.8 gpm per square foot of total area for the Mark 0 cloth and 4.5 gpm per square foot for the Mark I. These are based on a filterability index, I = 17.0, for both cloths corresponding to a suspended solids concentration of 25 mg/L in the influent for the Mark 0 cloth and 35 mg/L for the Mark I. The peripheral speed of the drum was 50 fpm.

At the Hanover plant of the Metropolitan Sanitary District of Greater Chicago (Ref. 7), a full size microstrainer with a diameter of 10 feet and a length of 10 feet was installed with Mark 0 cloth. This unit was built for a maximum head loss of 6 inches and a hydraulic loading of 6.6 gpm per square foot of total screen area equivalent to about 10 gpm per square foot of submerged area. The maximum solids loading of secondary effluent handled by the microstrainer was 0.88 psf per day equivalent to 11.1 ppm at the maximum flow of 6.6 gpm per square foot. On the other hand, the unit was able to handle a synthetic mixture of effluent, fortified with activated sludge at a rate of 3.8 psf per day, which is 6.8 gpm per square foot at 50 ppm solids concentration. This type of solids will filter much more readily than the pinpoint type of floc typical of secondary effluents. This fact must be kept in mind in applying the Callowhill results generally to the treatment of storm water overflows.

There are at least four manufacturers of this type of equipment using screen cloth with apertures of about 25 microns. The term "Microstrainer" is the copyrighted trade name of the Glenfield and Kennedy Division of the Crane Company. A scanning of the hydraulic capacities quoted in the trade literature yields figures varying from 4.6 to 28 gpm per square foot of total area, equivalent to 8 to 45 gpm per square foot of submerged area. Head losses range up to 30 inches, with 6 to 12 inches recommended. All literature contains statements to the effect that the capacity of the unit depends on the type of media (screen cloth) used, and the concentration and characteristics of the solids to be removed.

In sharp contrast to the Chicago and Lebanon data, the Callowhill results indicate that the microstrainer with Mark 0 cloth successfully handled suspended solids concentrations varying from 21 to 498 ppm and averaging 169 ppm. The peak flows and high concentrations were of short duration and the flows were not given. However, it was found possible to increase the flow per square foot fivefold by blanking off 80 percent of the area, thus increasing the differential head up to 30 inches. The actual flow rates for this condition were not given, but if both pumps were used, a flow rate of 47.5 gpm would have been possible. The explanation for the great difference in these results and those in Chicago and Lebanon would appear to be in the character of the suspended solids. For an area as small as 11 acres, the sewage would undoubtedly be fresh and would consist of discrete particles that had not had time to disintegrate. This would not be the case with large areas for which an adjustment factor is included in the model.

The above contrasts have made the sizing and performance estimates for the microstrainers difficult. The Callowhill data have been relied upon primarily but theoretical computations based on Chicago and Lebanon and other data have been used as a check. From Ref. 7 it was learned that the suspended solids removed would plot as a straight line versus the suspended solids in the influent. These data for Mark 0 cloth from

Table 1 of Ref. 8 have been plotted on Figure 15-3. The straight line, drawn by eye, has the equation

$$y = x - 35 \tag{11}$$

where y = Suspended solids removed (ppm)

x = Suspended solids in influent (ppm)

Since this equation would yield zero removal for influent suspended solids less than 35 ppm, it was restricted to suspended solids equal to or greater than 70 ppm. A parabolic curve having the equation

$$y = \frac{x^2}{140} \tag{12}$$

was inserted between the origin and x = 70. Eqs. 11 and 12 are incorporated in the Treatment model for determining the suspended solids removal of the flow through the screen cloth. They give no hint as to the capacity of the unit.

For determining capacity, other considerations were necessary. First, for economical reasons, as large a capacity as possible was desired, say 40 gpm per square foot of submerged area. This is about half the capacity of the 50 gpm per square foot of total area used for sizing the fine screens. It is four times the capacity of the Chicago unit and six times the capacity of the Lebanon unit, and approaches the maximum pumping rate possible at the Callowhill unit with the screen area reduced 80 percent. It was realized that this unit would have to be designed for a 30-inch differential head. It was also anticipated that the capacity might be reduced at high solids concentration with part of the

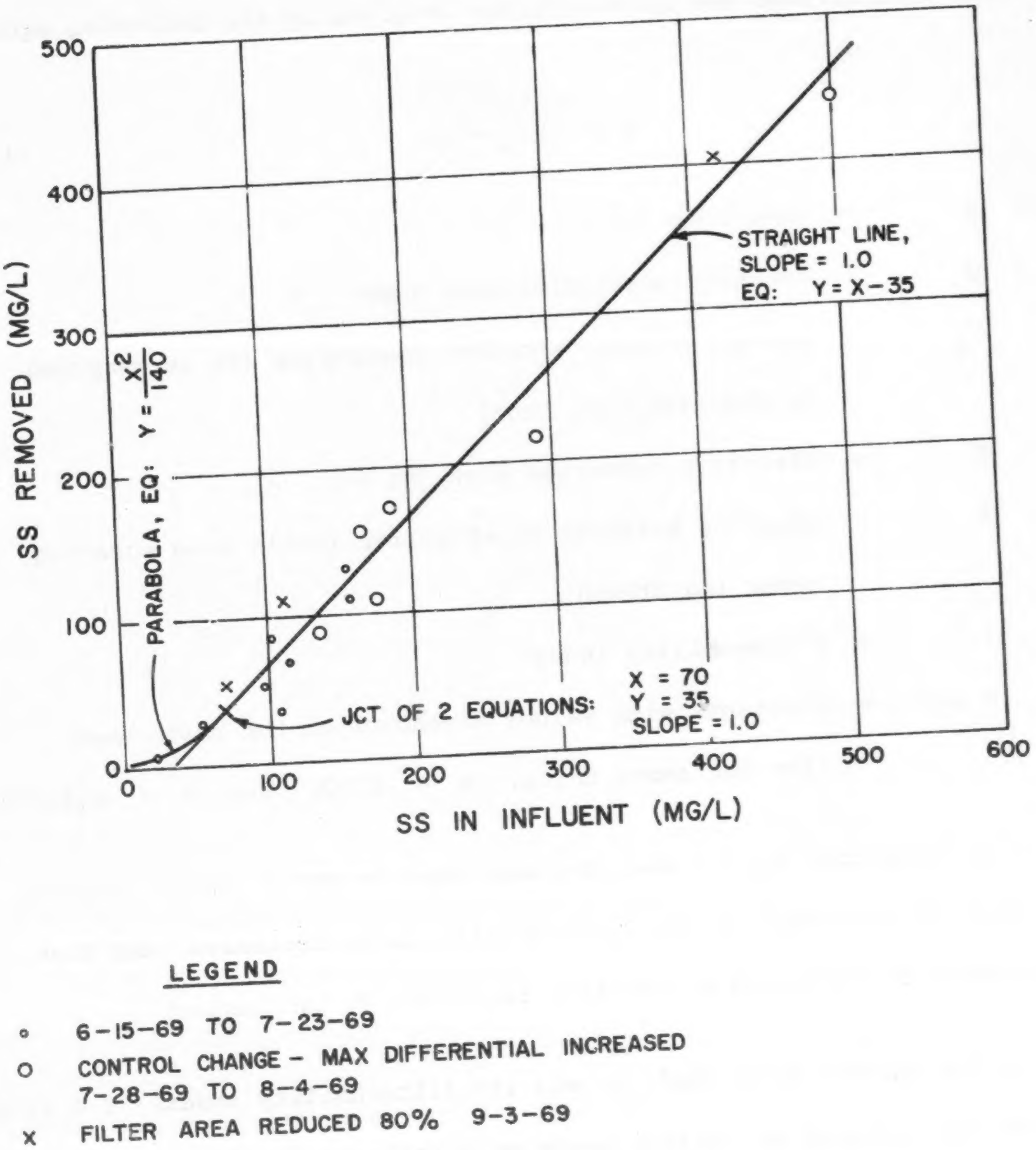


Figure 15-3. MICROSTRAINER AT CALLOWHILL (PHILADELPHIA), MARK O SCREEN CLOTH

flow bypassing the unit. The throughput of the unit should therefore be made dependent on the solids concentration and also a factor denoting the composition and condition of the solids.

The head loss through the microstrainer is given by the following equation:

$$H = \frac{mQC_f e^{nIQ/S}}{A} \qquad (13)$$

where H = Head loss (in.)

Q = Constant total flow rate (gpm)

$C_f$ = Initial (clean) strainer resistence (ft as measured in filterability test)

A = Effective submerged area (sq ft)

S = Speed of strainer in effective fabric area entering water (sq ft/min)

I = Filterability index

m and n = Constants with values dependent on the units used (for the above units, $m = 0.0267$ and $n = 0.1337$)

It will be seen from Eq. 13 that the head loss is not a simple function of Q, that it increases as the filterability index increases, and that it decreases as the speed of rotation increases (S increasing).

Based on the Lebanon data (Ref. 6) and its filterability index, $I = 17.0$, the flow was computed at various heads as a ratio to the flow $Q_1$ at a head loss of 6 inches. Then the index was reduced successively to 10, 5, 3, and 1 and the flow was computed at various head losses. The results are shown on Figure 15-4. The filterability is the reciprocal of the

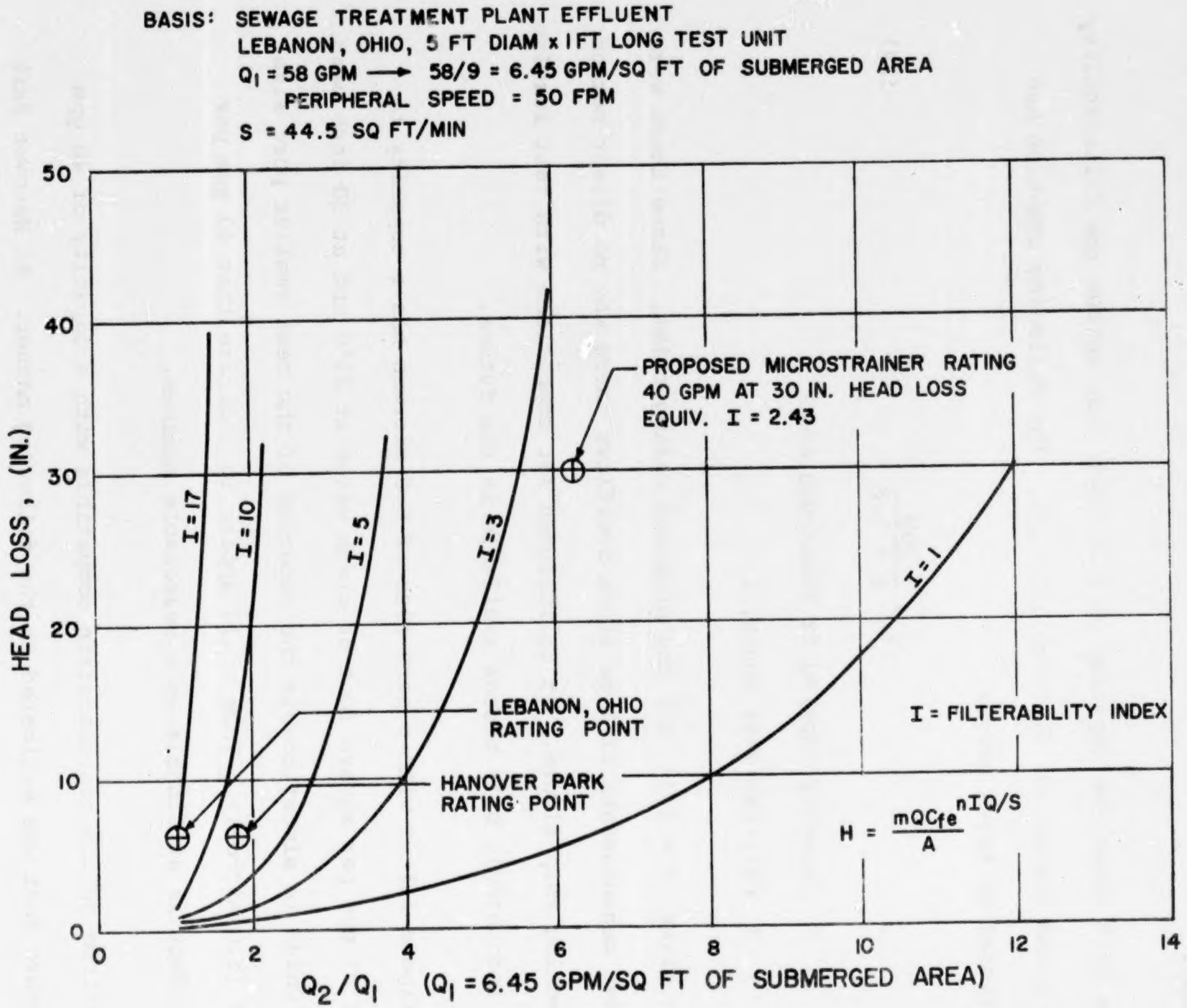


Figure 15-4. MICROSTRAINER CAPACITY, MARK O SCREEN CLOTH

filterability index, and it will be noted that as I decreases, the advantage of operation at greater heads increases. The proposed operating point for the Treatment model and for Chicago and Lebanon plants is shown.

Figure 15-5 shows the capacity at a 30-inch head versus the filterability index I and versus the filterability 1/I. The following equation has been fitted to these curves:

$$x = \frac{169}{y + 1.8} \tag{14}$$

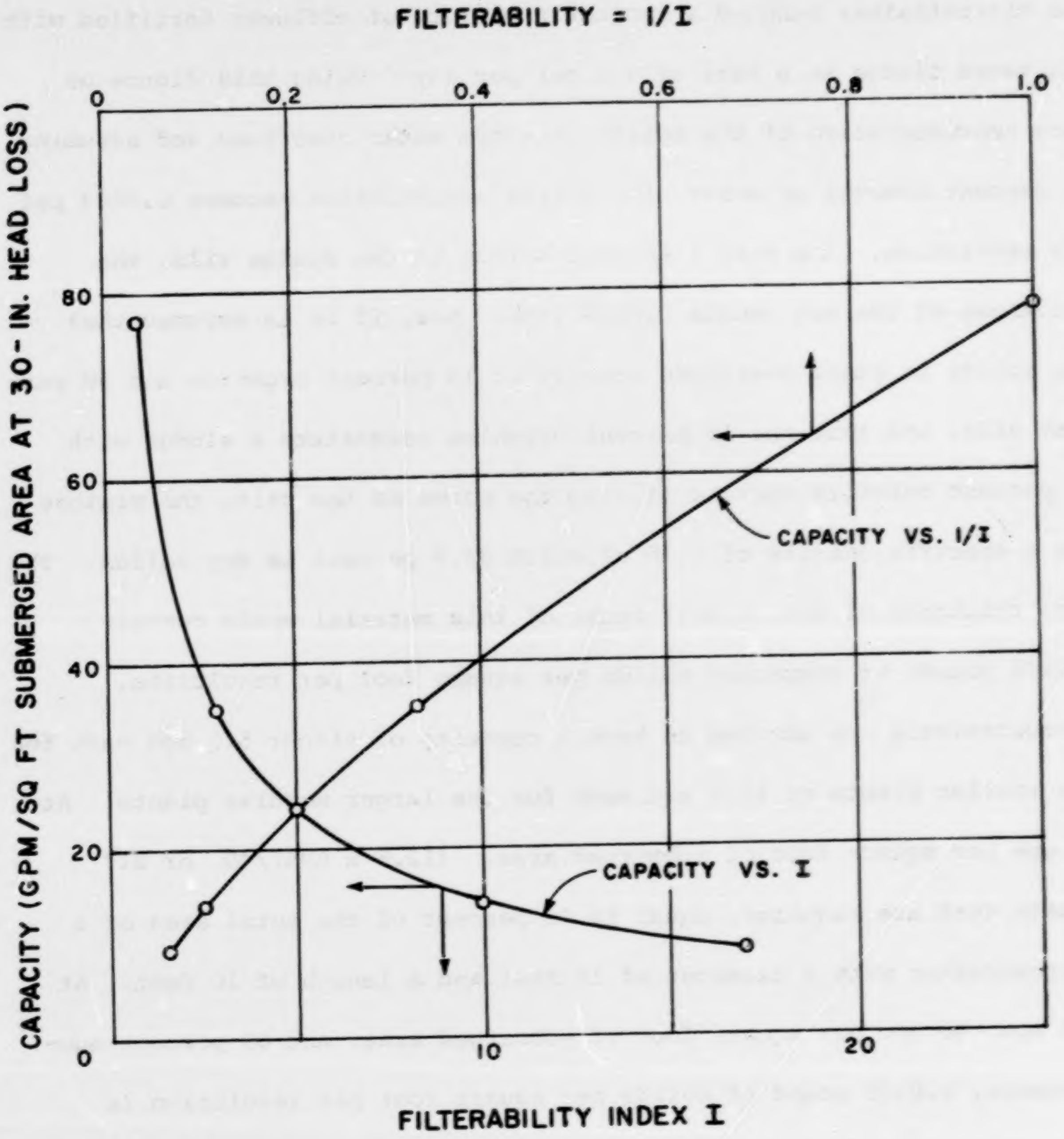
where x = Capacity (gpm/sq ft submerged area)

y = Filterability index, I

Eq. 14 gives I = 2.43 for the proposed rating point. Since there were no known measurements of I on storm overflows there was no direct method of checking this figure. It is included for comparison with test results which undoubtedly will become available in the future.

A millipore filter with a pore size of 8.0 microns has a capacity to filter 37 gpm per square foot of clean water at 25°C and at 30-inch head loss, which is almost double the capacity of the next smaller pore size filter (5.0 microns). This would appear to indicate that 40 gpm per square foot is attainable as a reasonable maximum.

The maximum solids concentration compatible with a capacity of 40 gpm per square foot was estimated in the following manner. At Hanover Park in Chicago the maximum solids loading of the applied effluent was 0.88 psf of total area per day, of which 65 percent was removed on the average.



FITTED EQUATION: $X = \frac{169}{Y + 1.8}$ WHERE

X = CAPACITY

Y = I

Figure 15-5. MICROSTRAINER CAPACITY AT 30-INCH HEAD LOSS, MARK 0 SCREEN CLOTH

This amounts to 0.000092 psf per revolution. However, as noted earlier, the microstrainer handled a synthetic mixture of effluent fortified with activated sludge at a rate of 3.8 psf per day. Using this figure as more representative of the solids in storm water overflows and assuming 65 percent removal as above, the solids accumulation becomes 0.0004 psf per revolution. Assuming 1 percent solids in the sludge film, the thickness of the mat equals 0.0077 inch. Now, if it is assumed that the solids in storm overflows consist of 10 percent organics and 90 percent silt, and that the 10 percent organics constitute a sludge with 96 percent moisture content filling the pores of the silt, the mixture has a specific gravity of 1.15 of which 29.4 percent is dry solids. The same thickness of mat, 0.0077 inch, of this material would contain 0.0135 pounds of suspended solids per square foot per revolution. Microstrainers are assumed to have a capacity of either 5.0 mgd each for the smaller plants or 12.5 mgd each for the larger modular plants. At 40 gpm per square foot of submerged area, (12.5 x 695)/40 or 217 square feet are required, equal to 69 percent of the total area of a microstrainer with a diameter of 10 feet and a length of 10 feet. At 4.3 rpm, 40 gpm per square foot of submerged area, and 69 percent submergence, 0.0135 pound of solids per square foot per revolution is equivalent to a removal of 252 mg/L. Multiplying this by 40 gpm per square foot gives 10,100, or say 10,000, as the product of gpm per square foot and suspended solids removal in mg/L. From this, the following equation for the capacity of the unit in terms of the removal of suspended solids in mg/L is obtained:

$$\text{gpm/sq ft submerged area} = \frac{10{,}000 \times F}{\text{SS removal in mg/L}} \qquad (15)$$

When the inflow suspended solids equals 287 ppm, Eq. 15, combined with the removal efficiency, gives a capacity of 40 gpm per square foot for F = 1.0. The model restricts the flow rate to not over 40 gpm per square foot of submerged area regardless of the solids concentration. If the solids concentration is so great that all the flow cannot be strained, part is bypassed and the overall performance is computed by the model.

F is introduced into Eq. 15 to allow for the effect of comminution and disintegration of the solids due to time of flow in the sewers. It has a maximum value of 1.0. The following is suggested for the value of F, subject to future pilot plant verification on large drainage basins, and is incorporated in the model:

$$F = \sqrt{\frac{400}{\text{area in acres}}} \qquad (16)$$

The value of F is suggested on the basis that no reduction would be necessary until the drainage basin exceeded 400 acres, and that then the reduction would be in proportion to the time of flow assumed proportional to the square root of the area.

The rate of backwash at Lebanon and at Chicago was 2.5 to 3.0 percent of the smaller flows treated, or approximately 50 gpm for a full size unit. It has been assumed that no greater quantity of wash water would be required, since this amounts to 5 gpm per linear foot of filter.

The backwash water rate has been rounded off to a constant flow of 0.1 cfs per microstrainer which will be pumped or will flow by gravity to a sanitary sewer or interceptor for transport to the sewage treatment plant.

With the operating performance determined on the basis of suspended solids, the remaining task was to compute the BOD removals. At Hanover Park, BOD removals were as good or even better than suspended solids removals, and at Lebanon they were about 90 percent of the suspended solids removal. On the other hand, the removals at Callowhill were erratic; many negative values were obtained. This may have been due to disintegration of the solids during pumping. The BOD removals at Callowhill cannot be used as a basis for the model.

On the other hand, excellent data were obtained at Callowhill on the removal of volatile suspended solids. BOD versus volatile suspended solids were plotted in the storm flows of the Laguna Street sewer in San Francisco for the storms of March 10 and March 15, 1967, reported by Engineering-Science (Ref. 9). The BOD was found to equal 80 percent of the volatile suspended solids. The removals of volatile suspended solids versus the volatile suspended solids in the influent were plotted as given in Table 3 of Ref. 8. The straight line equations obtained were multiplied by 0.8 to convert to BOD. The following equations were obtained and incorporated in the model:

BOD removed (ppm) = BOD in influent (ppm) - 10.0, if the BOD in the influent is equal to or greater than 27 ppm (17)

$$\text{BOD removed (ppm)} = 17.0 \times \text{BOD in influent (ppm)}/27.0, \text{ if the BOD in the influent is less than 27 ppm} \quad (18)$$

Due to the form of Eqs. 11, 12, 17, and 18, there would be no improvement in the suspended solids and BOD concentrations in the effluent from the microstrainers if the flow is given primary treatment in Level 3 unless the suspended solids and BOD concentrations in the influent to the microstrainers were reduced below 70 and 27 mg/L, respectively. However, in the case of heavy concentrations of suspended solids or large drainage basins, primary treatment in Level 3 could eliminate or greatly reduce the amount of flow bypassing the units, thus resulting in considerable overall improvement of the effluent.

High Rate Filters (Treatment Option 43)

Various types of rapid filters are being investigated for tertiary treatment of sewage plant effluents and for the treatment of overflows from storm and combined sewers. At Hanover Park in Chicago (Ref. 7) two Hardinge automatic backwash filters were installed to test their applicability for treating activated sludge plant effluent. They were operated in parallel with the microstrainer. Each filter contained 11 inches of sand, with an effective size of 0.58 millimeters and a uniformity coefficient of 1.62, supported on porous plates. These filters were operated with and without chemicals at a capacity of 2.5 gpm per square foot for a low head of 4.4 inches and at a capacity of 6 gpm per square foot for a high head of 11.5 inches. Suspended solids removals varied from 65 to 78 percent. The maximum solids removal at the higher rate was 0.65 psf per day equivalent to 9 ppm. The backwashing cycle takes 30 minutes to clean the whole bed, a narrow strip at a time.

For economical treatment of sewer overflows, considerably higher flow rates are desirable. Higher rates will require higher operating heads and coarser media. To obtain good efficiencies and large solids holding capacity, multiple media filters with a greater overall depth will probably be required. If the automatic backwash filter can be modified along the above lines to triple its capacity while maintaining good efficiency, it would be an ideal unit for this service.

In Washington, D.C., (Ref. 10) a synthetic storm overflow wastewater, developed by diluting domestic sewage and adding clays and silts to provide a waste comparable to combined sewer overflows analyzed at the time, was filtered through three laboratory filters. Each filter consisted of a 9-foot jointed glass pipe with an inside diameter of 4 inches. One filter contained fiber glass. A second filter contained 36 inches of anthracite, 24 inches of a garnet sand mixture, and 3 inches of coarse garnet supported on 9 inches of gravel. The third filter consisted of 48 inches of medium garnet and 9 inches of coarse garnet supported on 9 inches of gravel. It was designed and operated as an upflow filter, and performed reasonably well at flow rates between 5 and 15 gpm per square foot with suspended solids removals of 60 percent and BOD removals of 45 percent. At filtration rates exceeding 15 gpm per square foot the efficiency dropped sharply, as would be expected for this type of filter.

The second tri-media filter was operated as a downflow filter at a rate of 10 gpm per square foot. Runs lasted two hours with suspended solids removals of 80 to 95 percent and BOD removals of 50 to 80 percent. The higher rates were obtained with chemical additions, 150 mg/L of alum and 4 mg/L of flocculant aid. The same efficiencies were maintained at rates up to 20 gpm per square foot, but filter runs were shortened to one-half hour.

The fiber glass filter performed remarkably well at rates between 15 and 50 gpm per square foot with suspended solids removals in the range of 87 to 95 percent and BOD removals ranging from 60 to 75 percent; however, severe backwash problems were encountered. Runs lasted from one-half to one hour with 750 to 1,000 mg/L of suspended solids in the influent. Further tests of this filter should be made in a pilot plant of adequate size to determine design criteria and plant scale operating performance. Until such data are available, design estimates should be based on more conventional units, similar to the tri-media filter.

Pending further tests of the applicability of high rate filters to the treatment of storm and combined sewer overflows, a flexible program has been developed which will permit the engineer to select the following filter parameters:

1. Maximum flow rate, gpm per square foot
2. Maximum head loss
3. Holding capacity, pounds of dry solids per square foot at maximum rate and maximum head loss.

Initial values of these parameters are suggested in the following discussion and have been incorporated in the model.

Based on the tri-media filter described above, a design capacity of not more than 20 gpm per square foot and preferably less should be selected for sizing the units at the design flow. Each installation will contain an even number of units, not less than four. This will permit taking a unit out of service for backwashing. Maximum size of filter units will be 1,400 square feet, each with a maximum capacity of 40 mgd or 62 cfs.

A maximum head loss not greater than 10 feet is suggested.

Assuming that the water applied to the tri-media filter contained 750 mg/L of suspended solids of which 80 percent were removed and that a half-hour run at 20 gpm per square foot was feasible before backwashing, the solids removed by the filter amounted to:

$$20\ \frac{\text{gpm}}{\text{sq ft}} \times 30\ \text{min} \times 750\ \frac{\text{mg}}{\text{L}} \times 0.80 \times \frac{8.35}{10^6}\ \frac{\text{lb/gal.}}{\text{mg/L}} = 3.0\ \text{psf} \qquad (19)$$

It is suggested that this value or a lesser figure be taken as the holding capacity of the filter.

The suspended solids removal of the ripened filters has been taken at 80 percent without chemicals and 95 percent if coagulating chemicals are used. The BOD removal has been taken at 50 percent without chemicals and 80 percent with chemical additions. These removals are based on Ref. 10.

To allow for reduced efficiencies of clean filters, both efficiencies have been reduced by one-half when filters are first placed in service, and are allowed to increase uniformly with the pounds of suspended solids removed until 5 percent of the filter holding capacity has been accumulated, at which point the full efficiencies stated above are attained. Chemicals, if used, are computed at the rate of 150 mg/L of alum and 4 mg/L of flocculant aid based on Ref. 10.

Head losses will undoubtedly be high when operating at high rates such as these. For 24 inches of granular media with an effective size of 1.0 millimeters, the head losses at 68°F computed by Rose's method as given by Fair and Geyer (Ref. 4) are as follows:

| Gpm/sq ft | Head Loss, ft |
|---|---|
| 5 | 0.89 |
| 10 | 1.97 |
| 20 | 4.58 |

These losses do not include losses in the underdrain system which could be substantial. The head loss above varies approximately as the 1.18 power of the flow, and, on the basis of the above figures, it has been assumed that the head loss through a clean filter would be 40 percent of the maximum head at the design rate. At other rates of flow the head loss through a clean filter is given by the following equation:

$$H_{clean} = (q/q_m)^{1.18} \times .40\ H_m \qquad (20)$$

The head loss due to filter clogging is assumed to vary directly with the rate of flow and the solids accumulation:

$$\text{Head loss due to clogging} = \frac{q}{qm} \times \frac{S}{Sqm} \times (.60\ H_m) \qquad (21)$$

where S = Integrated sum of the solids removed in each time-step (psf)

Sqm = Solids holding capacity of the filter

The total head loss is equal to the sum of the head loss clean and the head loss due to clogging. When the total head with one unit out of service reaches 90 percent of the maximum head, the backwashing cycle is initiated and the filters are washed in rotation. This will allow for some head build-up on the last filters to be washed.

The backwashing of each filter, including taking out of service and placing back in service, is assumed to last one 10-minute time-step and to require an average backwash flow of 15 gpm per square foot over the 10 minutes. The backwash water will be discharged by gravity or pumped to a sanitary sewer or interceptor for transport to the sewage treatment plant.

When the first filter washed is placed back in service, integration of the solids accumulated on the filter begins anew. Efficiency of the clean filter is assumed to be reduced 50 percent for the first time-step; full efficiency is assumed for subsequent time-steps. The head loss on the first filter washed is allowed to reach maximum head, HM, the second time around before the washing cycle is again initiated, because each of the other filters will have been in operation a shorter length of time since they were cleaned.

WARNING: Later information in regard to the laboratory tests in Ref. 10, on which the program is based, indicates that the head loss was allowed to build up to 15 psi in most runs under which conditions the solids removed from the wastes were so compacted into the filter media that backwashing was time-consuming and difficult. It cannot be too strongly emphasized that the filter performance contained in the model has not been substantiated and remains purely speculative. There is no assurance that the filters will not plug up and overflow at design rates if solids concentrations are high. The model does not warn of or allow for this situation. High rate filters should not be proposed without pretreatment in Level 3. To do so is contrary to all experience with granular filters in the waterworks field.

Effluent Screens (Treatment Option 52)

Effluent screens are included for aesthetic improvement of overflows discharged at the shoreline or for treatment processes not including fine screening or filtering processes. No significant improvement in suspended solids or BOD can be computed for such installations.

Effluent screens were installed at the Rockaway high rate activated sludge plant of New York City to eliminate all traces of visible sewage solids which might pass through the plant and be discharged at the seawall. In many cases the aesthetic and psychological benefits, admittedly difficult to estimate, could be well worth the cost, especially if the installation of effluent screens made unnecessary the installation of more costly equipment. They are included in the Treatment model so that their cost can be estimated.

It is assumed that screens would be of the waterworks travelling basket type with 6-mesh wire. Two or more screens would be installed. They would have a capacity of 450 gpm per square foot based on the design flow. Assuming a maximum submerged area for each screen of 100 square feet, the maximum capacity per unit would be 100 cfs.

The volume of screenings removed is estimated at 0.05 cubic feet per million gallons. These would be removed by truck and disposed of in a sanitary landfill or at the municipal incinerator.

### Chlorination (Treatment Option 72)

Disinfection of storm and combined sewer overflows can be justified as a public health measure to protect downstream water supplies, bathing beaches, and other water uses. Chlorination is the most widely used method of disinfection and is the only method included in the Treatment model. While the decision whether or not to chlorinate will be based entirely on the need for disinfection, the addition of chlorine will result in an incidental reduction of BOD. Hence the Treatment model computes the reduction in both BOD and coliform organisms due to chlorination. Chlorination has no effect on suspended solids removals. The engineer must select the point of chlorine injection. Injection may be ahead of the sedimentation tanks or the dissolved air flotation tanks, which will serve as the chlorine contact tank if either sedimentation or dissolved air flotation is used. Otherwise it will be necessary to construct a chlorine contact tank. The volume of the tank is selected to provide a 15-minute detention period at the design flow.

It is assumed that the wastewater will be chlorinated to a residual. This requires estimation of the chlorine demand. The chlorine demand is estimated at 10 percent of the BOD content measured just ahead of the point of application (this figure is available in the Treatment model), but regardless of the value thus obtained, the chlorine demand is taken at not less than 6 ppm and not more than 25 ppm. The lower figure is based on Symons' work at Buffalo (Ref. 11) as reported by Camp (Ref. 12). The upper figure is based on the standards of the New York State Health Department for chlorination of primary effluents (Ref. 13). The number of chlorinators is equal to the chlorine demand in pounds per day, divided by 2,000 if the total demand is less than 8,000. If the total demand is greater than 8,000, it is assumed that chlorinators with a capacity of 8,000 pounds per day will be used, and the number is estimated by dividing the total demand by 8,000. If a chlorine contact tank is used the reduction of BOD is computed at 2.0 times the chlorine demand, in accordance with Ref. 14, but not more than 50 percent. If chlorine is applied ahead of sedimentation or dissolved air flotation the reduction in BOD has already been computed in a slightly different manner, but in a way that should yield comparable results.

Up to this point in the Treatment model there has been no discussion of the coliform content of the wastewater. With the application of chlorine for disinfection, however, the coliform content becomes of primary importance.

The coliform content of the wastewater is supplied by the Transport Model. The coliform content at the point of application of the chlorine is assumed to be reduced by the removal of suspended solids by previous treatment processes in accordance with the following formula:

$$\text{Coliforms at point of application} = \text{coliforms in influent} \times \frac{\text{SS at point of application}}{\text{SS at influent to treatment units}} \qquad (24)$$

The efficiency of chlorination is conservatively estimated to be 99.9 percent effective (Ref. 15). Coliform reductions are computed in subroutine KILL. If the detention period for chlorine contact in any time-step is less than 15 minutes, the efficiency is multiplied by $\frac{\text{detention period}}{\text{15 minutes}}$.

## SIZE OF DESIGN EVENT AND QUALITY OF TOTAL OVERFLOW RELEASED TO RECEIVING WATERS

At this time it is not possible to state what the size of the design event should be. It will be less than the maximum possible storm runoff because even the storm sewers are not designed for the maximum storm. For the sake of economy, except where public health is a consideration, it will probably be less than the runoff from a one-year storm. It may be different for drainage basins of different size and different locality. The ease with which the computer can compute results for events of different size will make it possible to compute the economic costs and benefits of varying design criteria. Experience with the program eventually will develop a better idea of the optimum size of the design event.

In any case, if the storms in any typical year are run through the model successively, the smaller storms will receive complete treatment, whereas the larger storms will produce some bypassing of the treatment units. In these cases the Treatment model will compute the content of suspended solids, BOD, and coliform organisms in the combined discharge to the receiving waters.

# PART 4

# ECONOMIC DATA

## SECTION 16

## COST-EFFECTIVENESS MODEL

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# SECTION 16

## COST-EFFECTIVENESS MODEL

### OBJECTIVE

The objective of the Cost-Effectiveness model is to illustrate the desirable size and type of treatment to be selected by providing:

1. Realistic costs of storage treatment for each storage option.
2. Realistic costs of treatment for each unit process.

The model is intended to provide guidance in the selection of the least costly combination of treatment processes and storage treatment sizes for varied storm patterns and locations.

### BACKGROUND

The effectiveness of treatment is computed in terms of removal efficiencies in the Treatment model and by the trace and the concentration histories of the waste field in the Receiving Water Quality model.

The cost model will enable the user to obtain associated costs with:

1. Each level of efficiency selected.
2. The efficiencies required to satisfy water quality standards.

The associated costs can be minimized by variation of the treatment and storage options selected. Automatic cost minimization, a desirable feature, was not within the scope of this study.

## THE MODEL SUBROUTINES

The internal storage costs are calculated by subroutine TSTCST within the internal storage model. Treatment costs and external storage costs are calculated by subroutine TRCOST within the Treatment model.

The operation of TSTCST and the internal storage model is governed by the Transport Block. The operation of TRCOST and the Treatment model is governed by the Storage Block.

### Subroutine TSTCST

Capital costs of hypothetical storage capacities are based on the maximum storage requirements.

The total cost is obtained from the fixed installation cost, and from the variable cost of reservoir capacity and the required land. Unit processes (see default values subsection) are used to compute storage and land costs.

### Subroutine TRCOST

Given a design flow (hence unit capacities), subroutine TRCOST determines the capital cost for the selected options (shown in Table 16-1) in the Treatment model. The cumulative operating costs are computed after simulation on each unit.

The sequence of operations in subroutine TRCOST is:

1. Read and summarize all unit costs and factors utilized in the computations.
2. Compute land and capital costs for each unit process selected and convert to annual costs.

Table 16-1. TREATMENT PROCESSES

---

Storage
Bar racks
Pumping stations (influent and effluent)
Sedimentation tanks
Fine screens
Microstrainers
Dissolved air flotation
High rate filters
Effluent screens
Chlorine contact tanks

---

3. Compute variable costs due to occurrence of storm events.
4. Present summary of all treatment costs.

Regional adjustments for the derived costs can be made by using the ratio of the individual city ENR (Engineering News-Record) index to the national average index. Also adjustments to future years may be made using projected values of the appropriate index. A set of suggested values for the years 1970 to 1980 is supplied (Volume III, Section 5). These future indexes were derived from the postwar ENR construction indexes.

## THEORETICAL DEVELOPMENT

When feasible the computed costs are based on generalized cost functions covering the range of treatment capacities (5 to 500 mgd).

The cost functions derived from one of the following forms, depending on the unit process selected:

$$\text{Unit cost} = aQ^{b} \tag{1}$$

$$\text{Unit cost} = cQ \tag{2}$$

$$\text{Log unit cost} = \frac{1}{a + b \log Q} \tag{3}$$

where Q = Design flow

a, b, c = Parameters to be derived

The preceding functions closely agree with the latest study on treatment process costs (Ref. 1).

The cost data utilized were adjusted for both variation in time and geographical location of the construction by using the ENR cost indexes.

Where data were not sufficient to derive a function, the unit costs were based on recent actual bid prices allowing for reduction in unit costs of larger installations. Further details on the basis of the individual treatment costs will follow.

The total treatment cost is based upon capital costs, operation and maintenance costs, and the cost of land.

## Capital Costs

The capital costs for each treatment process are calculated on the basis of maximum installed capacity determined by the "design flow."

The costs are converted to equivalent uniform annual costs using the supplied interest rate and the effective life of the installation.

## Operation and Maintenance Costs

In subroutine TRCOST operation and maintenance costs are divided into two parts: (1) the costs resulting from the existence of the plant, called irreducible costs, which are independent of the plant operation; and (2) the costs associated with storms, called storm event costs, which depend upon the quantity and quality of the influents.

## Land Costs

The user of the program may have more accurate data in terms of availability and cost of local land. However, in the absence of such information, a typical unit price is provided (see default values subsection).

Land costs are computed on the basis of the area required for each unit process. These costs have also been converted to equivalent annual costs using:

$$\text{Annual cost} = (L \times i)/100 \qquad (4)$$

where $L$ = Total land costs

$i$ = Percentage interest rate

## BASIS FOR INDIVIDUAL TREATMENT COSTS

## Capital Costs

The basis for the cost of individual treatment processes are briefly described below, followed by a presentation of the equations used

(Table 16-2). Examples of the computed costs appear in the test application subsection.

1. Storage

   The storage cost is based on a unit cost for reservoir construction allowing for the reduction of this cost for larger installations.

2. Bar Racks

   This cost is based on actual adjusted prices for screen units. Installation costs modified from Smith (Ref. 2) include the cost of screen chamber, overflow, bypass chamber, and Parshall flume.

3. Pumping Stations

   Capital costs of both influent and effluent pumping stations are based on cost files (Metcalf & Eddy cost files developed for internal use) which were obtained from actual construction costs.

4. Sedimentation Tanks

   The computed cost of sedimentation tanks is based on similar cost file data. The costs are derived for rectangular tanks and can be adjusted to variable surface loading.

5. Fine Screens

   The use of fine screens for sewage treatment in the United States is at the experimental stage. For example, the Stephan and Schaffer paper (Ref. 3) does not include any plant using fine screening.

## Table 16-2. TREATMENT COST SUMMARY

| Option | Derived Capital Cost Equation, Dollars | Applicable Capacity Range, mgd |
|---|---|---|
| **Bar Racks Supply:** | $1000(11 + \frac{S}{240})n(\frac{ENR}{1000})F$ | All |
| | where n = Number of screens<br>S = Screen capacity (cfs) -120, with a minimum of zero<br>ENR = ENR index for prescribed year (see Volume III)<br>F = Site factor (see Volume III) | |
| **Install:** | $10,000\ Q^{0.625}(\frac{ENR}{1034})F$ | $\leq$ 100 |
| | $1,780\ Q\ (\frac{ENR}{1034})F$ | > 100 |
| | where Q = Capacity (mgd) | |
| **Inlet Pumping** | $50,000\ Q^{0.58}(\frac{ENR}{1314})F$ | $\leq$ 20 |
| | $32,000\ Q^{0.73}(\frac{ENR}{1314})F$ | $20<Q\leq100$ |
| | $32,000\ Q\ (100)^{-0.27}(\frac{ENR}{1314})F$ | > 100 |
| **Dissolved Air Flotation** | $1.35\ Q\ \exp\left[\frac{2.3026}{0.2075+0.0114\ \mathrm{Log}Q}\right](\frac{ENR}{1098})F$ | $\leq$ 15 |
| | $1.35\ Q\ \exp\ [2.3026(4.6032-0.0559\mathrm{Log}Q)]\ (\frac{ENR}{1098})F$ | $15<Q<100$ |
| | $31,000\ (1.35)Q(\frac{ENR}{1098})F$ | > 100 |
| **Fine Screens** | $12,000\ Q(\frac{ENR}{1314})F$ | All |
| **Sedimentation In new tanks** | $43,000(\frac{700Q}{R})^{0.91}(\frac{ENR}{1000})F$ | < 100 |
| | $430\ Q\ (\frac{70,000}{R})^{0.91}(\frac{ENR}{1000})F$ | $\geq$ 100 |
| | where R = Overflow rate (gpd/sq ft ) | |
| **In storage** | $UV(\frac{ENR}{1314})F$ | All |
| | where U = Construction cost ($/cy)<br>V = Maximum storage during storm (cy) | |
| **Microstrainers** | $30,000\ Q(\frac{ENR}{1314})F$ | < 25 |
| | $20,000\ Q(\frac{ENR}{1314})F$ | $\geq$ 25 |

Table 16-2. TREATMENT COST SUMMARY (Continued)

| Option | Derived Capital Cost Equation, Dollars | Applicable Capacity Range, mgd |
|---|---|---|
| **High Rate Filters** | $54{,}000\ Q^{0.67}(\frac{ENR}{1058})F$ | All |
| **Effluent Screens**<br>**Supply and install:** | $5{,}000\ Q^{0.3}(\frac{ENR}{1314})F$ | $< 100$ |
| | $200\ Q\ (\frac{ENR}{1314})F$ | $\geq 100$ |
| **Channel works:** | $7{,}000\ Q^{0.625}(\frac{ENR}{1034})F$ | $< 100$ |
| | $1{,}246\ Q\ (\frac{ENR}{1034})F$ | $\geq 100$ |
| **Outlet Pumping** | (Same as Inlet Pumping) | |
| Chlorine Contact Tank & Equipment | $18{,}350\ Q^{0.628}(\frac{ENR}{1000})F$ | |
| | $16{,}000\ Q\ (\frac{ENR}{812})F$ | |

Two types of designs are envisioned. The first is based on a pilot plant installation at Seattle, Washington. The alternative design would use an installation identical to the microstraining process but with coarser screens. The capital costs derived are based on the latter alternative because of the wider use of such processes in Europe and the United States (two installations at Chicago, one at Lebanon, Ohio, and one at Denver). The capital cost of fine screens is directly comparable to microstrainers after allowance is made for increased capacities due to considerably larger open screen areas. A cost factor of 1/2 was used in this study.

6. Dissolved Air Flotation

Dissolved air flotation is also a relatively new process, hence it was felt that the most accurate basis for the derivation of capital costs would be the actual bid prices for a 25-mgd facility presently being constructed in San Francisco. The prices were modified to allow for:

a. Reduction in capital costs due to inclusion of units which were not an integral part of the process.

b. A 15 percent allowance for engineering and contingencies.

c. A 30 percent reduction in costs because of the difficult urban location of the site and possible conservative design capacity due to the untried nature of the treatment process.

d. Allowance was made for the variation in costs due to the size of the installation.

7. Microstrainers

Cost estimates for microstraining equipment were obtained from several manufacturers. However, when these costs (expressed per mgd of installed capacity) were compared to the actual bid prices for the few existing treatment plants, sharp variations in unit prices were obtained. Although available equipment costs can be considered reliable, installation costs per unit basis cannot. The primary reason for the inconsistency can be attributed to the large variations in installed capacities and uncertainty relative to the ultimate capacity of the microstraining units. Hence the unit prices that were used allow for larger design loading than is presently used and do not represent the price for any single installation.

8. High Rate Filters

These costs are based on cost file data and cost functions derived by Smith (Ref. 2). The costs are adjusted to allow for high surface loading of the filters.

9. Effluent Screens

These costs are based on several manufacturers' bid prices plus an allowance made for installation costs similar to bar rack installations.

10. Contact Tanks

The costs of contact tanks are based on function derived by Smith (Ref. 2). Separate costs of chlorinators are derived on the basis of cost file data.

## Irreducible Costs

For all unit processes the irreducible costs are assumed to be a fixed percentage of the initial capital investment. The amounts vary between 1 percent and 2 percent and depend upon the mechanical complexity of the equipment. Irreducible costs are presented in Table 16-3.

Table 16-3. IRREDUCIBLE MAINTENANCE COSTS

| Treatment Process | Assumed Percentage of Total Capital Investment Per Year |
|---|---|
| Bar screens | 1% |
| Pumping station, influent | 2 |
| Dissolved air flotation | 2 |
| Fine screens | 2 |
| Sedimentation tanks | 1 |
| Microstrainers | 2 |
| High rate filters | 2 |
| Effluent screens | 1.5 |
| Pumping station, effluent | 2 |
| Chlorine contact tanks | 2 |

## Storm Event Costs

The storm event costs are derived based on a fixed cost assumed to be associated with clean-up or check-up activity and a variable cost depending on the volume of the influent processed. These costs are summarized in Table 16-4.

Table 16-4. STORM EVENT COSTS

| Treatment Process | Assumed Variable Cost | Assumed Fixed Cost, $/storms |
|---|---|---|
| Bar racks | Based on the volume of solids to be disposed. | $15 |
| Pumping station | Based on 2¢/kwh and assumed efficiencies | 15 |
| Dissolved air flotation | 0.4¢/1,000 gal. | 15 |
| Fine screens | 0.6¢/1,000 gal. | 15 |
| Sedimentation tanks | 0.4¢/1,000 gal. | 15 |
| Microstrainers | 0.6¢/1,000 gal. | 15 |
| High rate filters | 1.0¢/1,000 gal. | 15 |
| Effluent screens | Based on the volume of solids to be disposed | 15 |
| Pumping station | Based on 2¢/kwh and assumed efficiencies | 15 |
| Chlorine contact tanks including chlorinators | Based on cost of chlorine 20¢/lb | 15 |

## DEFAULT VALUES

To assist the user, default values, shown in Table 16-5, are programmed in the cost subroutines.

## DATA LIMITATION

Several of the unit processes included in the Treatment model are advanced processes which have not been in operation beyond the pilot plant stage in the United States. As an example, a recent paper by Stephan and Schaffer (Ref. 3), giving an inventory of advanced treatment

Table 16-5. DEFAULT VALUES FOR COST SUBROUTINES

| Item | Value Programmed |
|---|---|
| Interest rate | 7% |
| Amortization period | 25 years |
| Site factor | 1.00 |
| Unit cost land | $20,000/acre |
| Unit cost power | 2¢/kwh |
| Unit cost chlorine | 20¢/lb |
| Unit cost polymers | $1.25/lb |
| Unit cost alum | 3¢/lb |

installations, cites only one plant using dissolved air flotation and three plants using microstrainers. Due to the newness of the processes, data concerning installation, operation and maintenance, and initial costs are necessarily limited. The unit costs used in the model should be continuously updated as more accurate data become available.

The cost effectiveness of the treatment alternatives should then be viewed in the light of this cost data limitation. Periodic revision of the cost model is highly desirable in order to provide a better decision making tool for the users of the Storm Water Management Model.

## TEST APPLICATIONS

Several runs selecting design flow, location, and actual storm patterns were made. The results are summarized in Table 16-6. Costs per acre of the tributary drainage basin, as well as total costs, are reported for ease of comparison of alternate solutions.

## Table 16-6. SAMPLE TEST RUNS

Location Smithville, U.S.A.
Tributary area 500 acres
Design flow 25 mgd
Total volume of influents processed 12 million gal.

Treatment options
Bar racks
Dissolved air flotation

```
                      CAPITAL    COSTS              ANNUAL    COSTS                     STORM  EVENT  COSTS
   TREATMENT     LEVEL     INSTAL       LAND       INSTAL       LAND     MIN MAINT   CHLORINE     CHEM      OTHER
    BAR RACKS      1      141915.      1293.      12178.        91.       1419.         0.         0.        25.
NO INLET PUMPING   2          0.          0.          0.         0.          0.         0.         0.         0.
DISS AIR FLOAT'N   3     1549642.      3581.     132976.       254.      30993.        35.       257.        23.
 BYPASS LEVEL 4    4          0.          0.          0.         0.          0.         0.         0.         0.
NO EFFL. SCREENS   5          0.          0.          0.         0.          0.         0.         0.         0.
NO OUTLET PUMPS    6          0.          0.          0.         0.          0.         0.         0.         0.
NO CONTACT TANK    7          0.          0.          0.         0.          0.         0.         0.         0.
               SUBTOTAL $ 1691557. $    4874. $  145154. $     341. $   32412. $      35. $     257. $     48.
               TOTAL        $ 1696431.                   $ 177907.                        $    339.
               TOTAL PER
               TRIB ACRE    $    3393.                   $    356.                        $      1.

               TOTAL LAND REQUIREMENT =      0.24 ACRES.
```

---

Location Smithville, U.S.A.
Tributary area 500 acres
Design flow 400 mgd
Total volume of influents processed 13 million gal.

Treatment options
Bar racks
Effluent screens

```
                      CAPITAL    COSTS              ANNUAL    COSTS                     STORM  EVENT  COSTS
   TREATMENT     LEVEL     INSTAL       LAND       INSTAL       LAND     MIN MAINT   CHLORINE     CHEM      OTHER
    BAR RACKS      1     1071662.      2092.      91960.       146.      10717.         0.         0.        48.
NO INLET PUMPING   2          0.          0.          0.         0.          0.         0.         0.         0.
 BYPASS LEVEL 3    3          0.          0.          0.         0.          0.         0.         0.         0.
 BYPASS LEVEL 4    4          0.          0.          0.         0.          0.         0.         0.         0.
EFFLUENT SCREENS   5      816943.      2092.      70102.       146.      12254.         0.         0.        16.
NO OUTLET PUMPS    6          0.          0.          0.         0.          0.         0.         0.         0.
  CONTACT TANK     7    11860430.     29406.    1017751.      2058.     237209.       135.         0.        15.
               SUBTOTAL $13749030. $   33590. $ 1179813. $    2351. $  260179. $     135. $       0. $     79.
               TOTAL        $ 13782620.                  $ 1442343.                       $    214.
               TOTAL PER
               TRIB ACRE    $   27565.                   $   2885.                        $      0.

               TOTAL LAND REQUIREMENT =      1.68 ACRES.
```

---

Location Smithville, U.S.A.
Tributary area 500 acres
Design flow 10 mgd
Total volume of influents processed 13 million gal.

Treatment options
Bar racks
Sedimentation
Microstrainers

```
                      CAPITAL    COSTS              ANNUAL    COSTS                     STORM  EVENT  COSTS
   TREATMENT     LEVEL     INSTAL       LAND       INSTAL       LAND     MIN MAINT   CHLORINE     CHEM      OTHER
    BAR RACKS      1       94475.      1261.       8107.        88.        945.         0.         0.        22.
NO INLET PUMPING   2          0.          0.          0.         0.          0.         0.         0.         0.
 SEDIMENTATION     3      310310.      8815.      26628.       617.       3103.         0.         0.        18.
 MICROSTRAINERS    4      343560.     11017.      29481.       771.       6871.         0.         0.        20.
NO EFFL. SCREENS   5          0.          0.          0.         0.          0.         0.         0.         0.
NO OUTLET PUMPS    6          0.          0.          0.         0.          0.         0.         0.         0.
NO CONTACT TANK    7          0.          0.          0.         0.          0.         0.         0.         0.
               SUBTOTAL $  748345. $   21093. $   64216. $    1477. $   10919. $       0. $       0. $     60.
               TOTAL        $   769438.                  $   76611.                       $     60.
               TOTAL PER
               TRIB ACRE    $    1539.                   $    153.                        $      0.

               TOTAL LAND REQUIREMENT =      1.05 ACRES.
```

NOT REPRODUCIBLE

# PART 5

# ACKNOWLEDGMENTS, REFERENCES, PUBLICATIONS, GLOSSARY AND ABBREVIATIONS, AND APPENDICES

# SECTION 17

# ACKNOWLEDGMENTS

## SECTION 17

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# SECTION 18

# REFERENCES

## SECTION 18

## REFERENCES

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# SECTION 19

# PUBLICATIONS

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## PUBLICATIONS

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# SECTION 20

# GLOSSARY AND ABBREVIATIONS

# SECTION 20

## GLOSSARY

WATERSHED - The area which is drained by a river system.

DRAINAGE BASIN (STUDY AREA) - The area which contributes runoff to a stream at a given point (an individual section of a watershed).

SUBCATCHMENT - A subdivision of a drainage basin (generally determined by topography and pipe network configuration).

SUBAREA - A subdivision of a subcatchment (generally based upon a single land use but may be identical to a subcatchment).

## ABBREVIATIONS

| | |
|---|---|
| APWA | - American Public Works Association |
| ASCE | - American Society of Civil Engineers |
| EPA | - Environmental Protection Agency |
| M&E | - Metcalf & Eddy, Inc. |
| UF | - University of Florida |
| USPH | - U.S. Public Health Service |
| WRE | - Water Resources Engineers, Inc. |

| | |
|---|---|
| BOD | - biochemical oxygen demand (5-day) |
| cf | - cubic feet |
| cfs | - cubic feet per second |
| COD | - chemical oxygen demand |
| DO | - dissolved oxygen |
| DWF | - dry weather flow |
| fpm | - feet per minute |

fps - feet per second

ft - feet

gal. - gallons

gal./capita/day - gallons per capita per day

gpd - gallons per day

gph - gallons per hour

gpm - gallons per minute

gpm/sq ft - gallons per minute per square foot

gpsf - gallons per square foot

hr - hour

in. - inches

in./hr - inches per hour

JCL - job control language

lb - pounds

lb/acre/day - pounds per acre per day

lb/acre/yr - pounds per acre per year

lb/capita/day - pounds per capita per day

lb/cf - pounds per cubic foot

lb/day/cfs - pounds per day per cubic feet per second

lb/ft - pounds per foot

lb/sec - pounds per second

mgd - million gallons per day

mg/gram - milligrams per gram

mg/L - milligrams per liter

min - minutes

mm - millimeters

| | |
|---|---|
| MPN | - most probable number |
| ppm | - parts per million |
| psf | - pounds per square foot |
| psi | - pounds per square inch |
| rpm | - revolutions per minute |
| sec | - second |
| sq ft | - square feet |
| sq ft/min | - square feet per minute |
| SS | - suspended solids |
| tons/mo | - tons per month |
| tons/sq mi/mo | - tons per square mile per month |
| VSS | - volatile suspended solids |
| yr | - year |

## SYMBOLS

| | |
|---|---|
| Δ | delta |
| α | alpha |
| Σ | sigma |
| < | less than |
| > | greater than |
| ∂ | partial differentiation |
| ρ | rno |
| Ψ | psi |
| Π | pi |
| Θ | theta |

# SECTION 21

## APPENDICES

A. PRELIMINARY INFILTRATION ANALYSIS

B. DETAILED THEORETICAL DEVELOPMENT FOR DECAY MODEL

SEWER QUALITY ROUTING

General Equation for Continuity of Mass

Assumptions and Properties of the Various Hydraulic Elements in the Transport Model

SEDIMENT UPTAKE AND DEPOSITION MODEL

Procedure for Suspended Solids

Definition of Variables

Hydraulic Radius (Function RADH)

## APPENDIX A

## PRELIMINARY INFILTRATION ANALYSIS

It was originally thought that a generalized predictive equation for infiltration could be developed. By incorporating both spatially and time dependent variables, the generalized equation could then be applied to predicting sewer infiltration in any urban area in the country for any time of the year. The following summarizes the development of such an equation and the reasons for reverting to the analysis described in Section 7. (References cited are all in Section 7 of References).

One reason that so little is known of the importance and effect of infiltration on sewer flow is the lack of actual flow measurements of infiltration. This lack of flow data posed a severe constraint on justifying, developing, and verifying any theoretical or empirical Infiltration Model. To overcome this constraint, minimum daily sewage flow data recorded over a two-year period on the Johns Hopkins Residential Sewerage Research Project (Ref. 1)* for seven areas throughout the country were obtained. Because of the inclusion of foundation drains in at least one of the seven areas and the possibility of some sewage flow, the data give a slightly inflated estimate of infiltration.

Additional data were sought to supplement the above data since it has been noted that infiltration has been shown (Refs. 6,5) to vary with conditions of the sewer, level of groundwater, amount of precipitation, and soil conditions. By obtaining appropriate sewer, soil, and precipitation information, a sufficient data base was established. Categories and sources of information gathered are as follows:

1. Climatological data - U. S. Weather Bureau
2. Groundwater data - U. S. Geological Survey
3. Sewer system data - individual cities.

In addition, the following manufacturers of sewer pipe were contacted:

1. National Clay Pipe Institute
2. Cast Iron Research Association
3. Dickey Clay Mfg. Company
4. Portland Cement Association
5. American Concrete Pipe Association
6. Hamilton Kent Mfg. Company
7. K. T. Snyder Company, Inc.
8. American Pipe Services.

Multiple regression techniques were then applied to the data to describe daily variation in the recorded minimum flows. Flow variance was initially divided into that due to either location-dependent or time-dependent variables with a separate analysis performed for each. Location-dependent variables describing the seven study areas were correlated with three-month average minimum flows using the following procedure:

1. Determination of the existence of regression using three-month averages for the following study area data:
    a. Number of service connections
    b. Area
    c. Distance between manholes

   - d. Pipe diameter
   - e. Total pipe length
   - f. Pipe grade
   - g. Jointing material
   - h. Age
   - i. Pipe material
   - j. Water table head.

2. Elimination of variables accounting for the least amount of variance.
3. Elimination of redundant variables.
4. Transformation of the remaining variables to obtain better correlation and regression.

From the above data manipulations, total pipe length, pipe diameter, jointing compound, and water table height above the sewer accounted for 95 percent of the variance in minimum sewage flows. Water table height was taken as an average and squared to increase correlation. Jointing compounds were qualified by noting their relative susceptibility to root penetration. From the amount of sewer clogging due to root penetration in the seven study areas, relative factors for joints were established:

1. Cement mortar, 11.0
2. Lime mortar, 9.0
3. Bituminous hot pour, 3.5
4. Rubber gasket, 1.0.

After receiving necessary climatological and geological information for each of the seven study areas, analysis of variance was done on time-dependent variables and minimum flows. Data manipulation over time was as follows:

1. Determination of the existence of regression using the following recorded data:
    a. Rainfall
    b. Groundwater level
    c. Temperature
    d. Relative humidity.
2. Elimination of variables accounting for the least amount of variance.
3. Transformation of remaining variables to obtain better correlation and regression as follows:
    a. Use of time delays ranging from one to six days on rainfall and water table data.
    b. Use of power transformation on water table data.
    c. Use of a precipitation index to indicate soil moisture conditions.

From the above data manipulations, only rainfall significantly accounted for minimum flow variation in each study area. Transformations on rainfall data using both a time delay depending on soil type and an antecedent precipitation index (Ref. 8) were proposed and tested successfully. Time delays were found by regressing rainfall from each of six previous days on daily minimum sewage flows. From data from four of the seven study areas, rain falling on the indicated soil type and the indicated number of days prior to minimum flow measurement correlated best as follows:

1. Day 3 for sandy soil
2. Day 4 for rocky soil
3. Day 5 for clay soil.

Soil moisture was accounted for with the antecedent precipitation index found by applying Eq. A1 to prior daily rainfall.

$$\mathrm{API} = b_1P_1 + \ldots + b_tP_t + \ldots + b_nP_n \tag{A1}$$

where $P_t$ = Amount of precipitation (in.) which occurred t days prior to flow measurement

$b_t$ = $K^t$ with

$K$ = 0.86 to 0.89 for eastern and central portions of country

0.76 for high precipitation portions of country

0.94 for low precipitation portions of country

Remaining variables from the above analyses were finally combined in an overall regression analysis to form a predictive equation for infiltration that depends upon subarea characteristics and time. Eq. A2 accounted for 83 percent of the variation in the Johns Hopkins data and exhibited significant regression at the 95 percent confidence level.

$$\begin{aligned}\mathrm{QINFIL} = {} & -20.64 + 15.41(\underline{\mathrm{API}}) + 2.79(\underline{\mathrm{DIAM}}) - 31.71(\underline{\mathrm{RNY}}) + \\ & 0.64(\underline{\mathrm{CJOIT}}) + 0.25(\underline{\mathrm{LEN}})\end{aligned} \tag{A2}$$

where API = Antecedent precipitation index

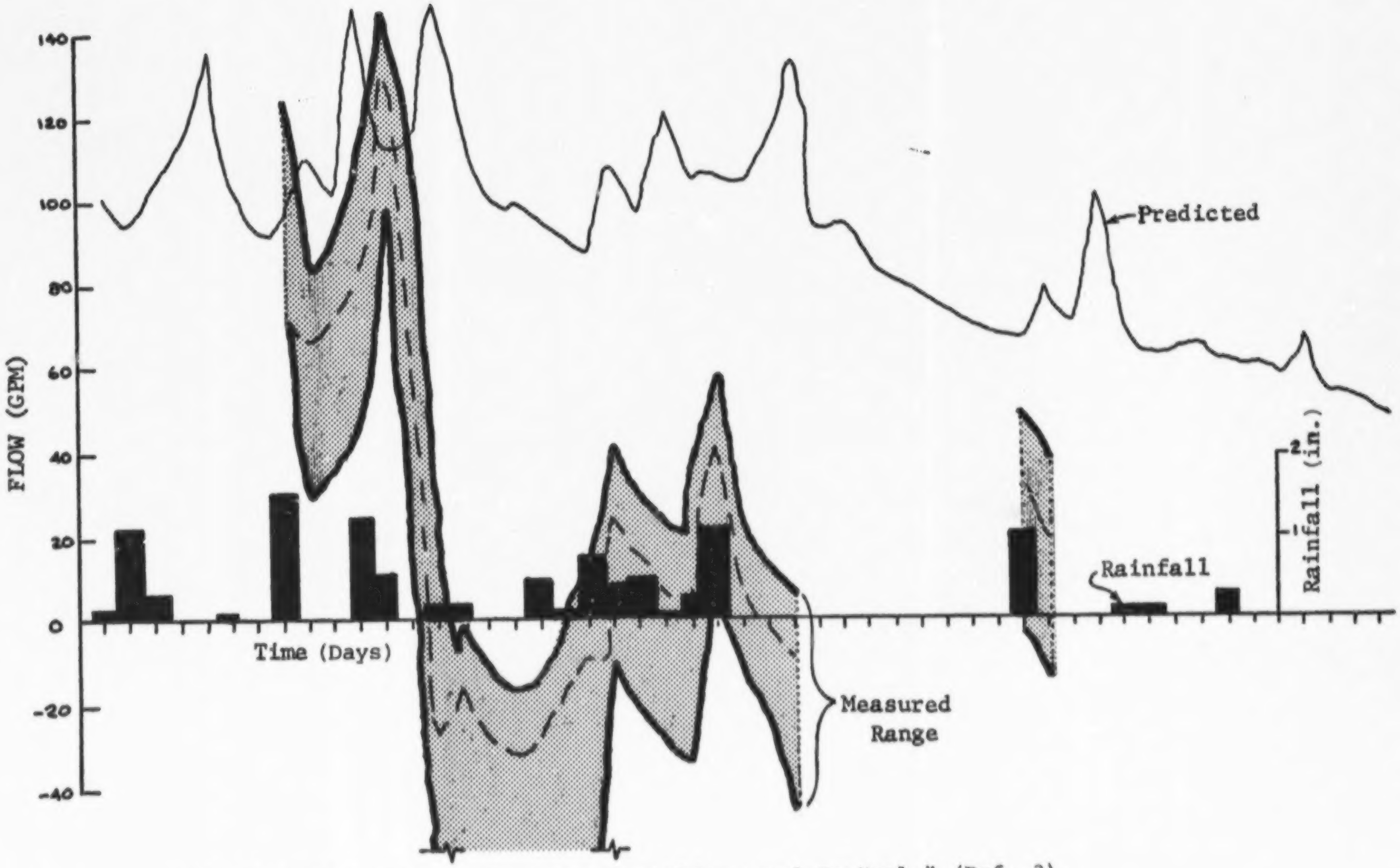
DIAM = Diameter of largest subarea pipe (in.)

RNY = Rainfall Y days prior to infiltration estimate (in.), Y = 3 for sandy soil, Y = 4 for rocky soil, and Y = 5 for clay soil

CJOIT = Joint factor

LEN = Total sewer length in each subarea (miles)

By showing significant regression, Eq. A2 satisfied the initial assumption that infiltration could be modeled using information on the availability, movement, and entry of moisture into sewers. Availability of moisture is included in the two-time dependent variables, API and RNY. In addition, RNY introduces a time delay due to soil conditions affecting movement of soil moisture. Although important in preliminary analyses, water table height above the sewer was eliminated in the final regression analysis for Eq. A2 because of very low correlation. The variables LEN, CJOIT, and DIAM control moisture's ability to enter sewer pipe. Results of testing the preliminary infiltration equation on three subareas studied by M&E were extremely helpful in testing the equation. Specific results from the comparison (see, for example, Figure A-1 of the Glen Street subarea, Berkeley, California) were that the preliminary predictions consistently exceeded corresponding measurements on an average of 80 gpm (0.18 cfs) and failed to account for abrupt flow changes. To account for these discrepancies, additional developments led to the approach described in Section 7.



Source: Metcalf & Eddy, Inc., "Storm Water Problems and Control," (Ref. 3).

Figure A-1. COMPARISON OF PREDICTED AND MEASURED INFILTRATION FLOWS M&E STUDY, GLEN STREET (BERKELEY, CALIFORNIA) SUBAREA

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# APPENDIX B

## DETAILED THEORETICAL DEVELOPMENT FOR DECAY MODEL

### SEWER QUALITY ROUTING

### General Equation for Continuity of Mass

$$\begin{aligned}\begin{matrix}\text{Pounds in}\\ \text{element at}\\ \text{new time-step}\end{matrix} &= \begin{matrix}\text{Pounds in}\\ \text{element at}\\ \text{old time-step}\end{matrix} + \begin{matrix}\text{Pounds}\\ \text{entering}\end{matrix} - \begin{matrix}\text{Pounds}\\ \text{leaving}\end{matrix} \pm \\ &\quad \begin{matrix}\text{Pounds}\\ \text{decayed or}\\ \text{generated}\end{matrix} \pm \begin{matrix}\text{Pounds entering or}\\ \text{leaving from}\\ \text{source or sink}\end{matrix}\end{aligned} \tag{B1}$$

Notation for Eq. B1

$C$ = Conc. in element (lb/cf)

$\bar{V}$ = Volume in element (cf)

$n$ = Time-step

$A$ = Cross-sectional area (sq ft)

$L$ = Length of conduit (ft)

SURGE = Volume stored during surcharging (cf)

$D_1$ = Decay rate

$D_2$ = Growth rate

$S$ = Maximum growth

$\Delta t$ = Time increment

WELL = Volume stored in lift station wet well

Expanding Eq. B1 with the above notation yields:

$$\begin{aligned}(C\bar{V})_{n+1} &= (C\bar{V})_n + \frac{(C_{in}\bar{V}_{in})_n + (C_{in}\bar{V}_{in})_{n+1}}{2} - \frac{(C_{out}\bar{V}_{out})_n + (C_{out}\bar{V}_{out})_{n+1}}{2} \\ &\quad - \Delta t\, D_1 \frac{(C\bar{V})_n + (C\bar{V})_{n+1}}{2} + \Delta t\, D_2 \frac{(S-C_n)\bar{V}_n + (S-C_{n+1})\bar{V}_{n+1}}{2}\end{aligned} \tag{B2}$$

By assuming complete mixing

$$(C_{out})_{n+1} = C_{n+1} \text{ and } (C_{out})_n = C_n \qquad \text{(B3)}$$

and substituting Eq. B3 into Eq. B2 and collecting terms,

$$C_{n+1}\left[\bar{V}_{n+1} + \frac{(\bar{V}_{out})_{n+1}}{2} + \Delta t\ D_1\frac{\bar{V}_{n+1}}{2} + \Delta t\ D_2\frac{\bar{V}_{n+1}}{2}\right] =$$

$$C_n\left[\bar{V}_n - \frac{(\bar{V}_{out})_n}{2} - \Delta t\ D_1\frac{\bar{V}_n}{2} - \Delta t\ D_2\frac{\bar{V}_n}{2}\right] + \qquad \text{(B4)}$$

$$\frac{(C_{in}\bar{V}_{in})_n + (C_{in}\bar{V}_{in})_{n+1}}{2} + \Delta t\ D_2\ S\ \frac{\bar{V}_n + \bar{V}_{n+1}}{2}$$

Since

$$\frac{(\bar{V}_{out})_{n+1}}{\Delta t} = (Q_{out})_{n+1} \quad \text{and} \quad \frac{(\bar{V}_{out})_n}{\Delta t} = (Q_{out})_n \qquad \text{(B5)}$$

Multiplying by 2/Δt Eq. B4 then becomes:

$$C_{n+1}\left[\bar{V}_{n+1}\ (2/\Delta t + D_1 + D_2) + (Q_{out})_{n+1}\right] =$$

$$C_n\left[\bar{V}_n\ (2/\Delta t - D_1 - D_2) - (Q_{out})_n\right] + \qquad \text{(B6)}$$

$$\frac{(C_{in}\bar{V}_{in})_n}{\Delta t} + \frac{(C_{in}\bar{V}_{in})_{n+1}}{\Delta t} + D_2\ S\ (\bar{V}_n + \bar{V}_{n+1})$$

Solving for $C_{n+1}$ yields:

$$C_{n+1} = \Big\{C_n\left[\bar{V}_n\left(\frac{2}{\Delta t} - (D_1 + D_2)\right) - (Q_{out})_n\right] + (C_{in}Q_{in})_n +$$

$$(C_{in}Q_{in})_{n+1} + D_2\ S(\bar{V}_n + \bar{V}_{n+1})\Big\} \Big/ \Big[V_{n+1}\left(\frac{2}{\Delta t} + (D_1 + D_2)\right) + \qquad \text{(B7)}$$

$$(Q_{out})_{n+1}\Big]$$

Eq. B7 becomes Eq. B8 by the following notation:

$n$ = Old time-step, 1

$n+1$ = New time-step, 2

$j$ = Upstream point, 1, of element M

$j+1$ = Downstream point, 2, of element M

VOL1 = Mixing volume, $\bar{V}_n$

VOL2 = Mixing volume, $\bar{V}_{n+1}$

DT = $\Delta t$

CPOLL = C

CPOLL(M,2,1) = Concentration of a given pollutant in element M, downstream and at the old time-step

$$\text{CPOLL}(M,2,2) = \left\{\text{CPOLL}(M,2,1)\left[\text{VOL1}\left(\frac{2}{\text{DT}} - (D_1 + D_2)\right) - Q(M,2,1)\right] + D_2\,(S)\,(\text{VOL1} + \text{VOL2}) + \text{CPOLL}(M,1,1)Q(M,1,1) + \text{CPOLL}(M,1,2)Q(M,1,2)\right\} \Big/ \left[\text{VOL2}\left(\frac{2}{\text{DT}} + (D_1 + D_2)\right) + Q(M,2,2)\right] \tag{B8}$$

## Assumptions and Properties of the Various Hydraulic Elements in the Transport Model

1. Conduits: L = length, A = area

$$\text{VOL1} = L\,\frac{(A_{j,n} + A_{j+1,n})}{2}$$

$$\text{VOL2} = L\,\frac{(A_{j,n+1} + A_{j+1,n+1})}{2}$$

2. Manholes and Flow Dividers

VOL1 = SURGE1(M)

VOL2 = SURGE2(M)

3. Lift Stations

VOL1 = WELL1(M)

VOL2 = WELL2(M)

4. Storage Units

Quality routing is accomplished according to the procedure given in the Storage Model.

## SEDIMENT UPTAKE AND DEPOSITION MODEL

The objective is to determine for each conduit in a general sewer system the quantity of sediment uptake and deposition under DWF and/or storm conditions. Given input pollutographs for suspended solids and DWF, for each time increment DT, let the average velocity of flow, V, needed to transport particles of diameter d in a sewer element, M, be defined as:

$$V = \frac{1.486}{N} R^{1/6}\sqrt{k(S_s-1)d} \qquad \text{(B9)}$$

where N = Manning's N for roughness

R = Hydraulic radius

d = Particle diameter

k = Shields' magnitude of sediment characteristic

$S_s$ = Specific gravity of particle

Using Manning's formula for V we have

$$V = \frac{1.486}{N} R^{2/3} (S')^{1/2} \qquad \text{(B10)}$$

in which N = Manning's N for roughness

R = Hydraulic radius

S' = Slope of invert of pipe (hydraulic energy line)

Setting Eqs. B9 and B10 equal and solving for d, then

$$d = \frac{RS'}{k\,(S_s - 1)} \qquad \text{(B11)}$$

The above procedure is used to solve for the particle diameter, d, corresponding to the velocity in the sewer conduit. For example, given the velocity (V) at any element in the sewer system for a specific time increment (DT), there is a critical particle diameter (CRITD) related to sediment uptake and a CRITD for deposition.

For sediment uptake, Case 1, that portion of sewer bed load that has a particle diameter (PD) equal to or greater than the critical particle diameter (PD ≥ CRITD) will remain at the bottom of the sewer. All particles with PD < CRITD will be transported. When dealing with deposition, Case 2, particles with PD ≥ CRITD will settle to the bottom of the sewer element while particles with PD < CRITD will be transported down the element.

A sieve analysis curve is used to describe sediment characteristics within a sewer. This curve was developed from data obtained in the APWA report (Ref. 8, Section 13). The data were averaged to represent a typical sieve analysis curve for sewer sediment. The curve now being used in QUAL is shown in Figure B-1. Three straight lines are used to approximate the plot. However, if actual sieve analyses of sewer sediment are taken, this information should be used to replace the present curve. Also, an updating of the curve for Case 1 and Case 2 should be considered along with a more detailed breakdown on suspended solids.

Eq. B11 is used to determine CRITD. From the information available on sewer sediment, an average $S_s$ of 2.7 is assumed. Shields' value for k is taken to equal .056 and the values for S are obtained from the Transport Model. The procedure used in calculating R is presented in function RADH, the last section in this appendix.

Procedure for Suspended Solids

Step 1. Calculate average velocity in each conduit. This is general information since V is not used in Eq. B11.

Step 2. Calculate hydraulic radius as shown in function RADH.

Step 3. Determine CRITD for Case 1 and CRITD for Case 2 using Eq. B11.

Step 4. Determine PCT1 and PCT2 using the equations representing Figure B-1.

Step 5. Use the following equations to determine sediment uptake and deposition:

$$\text{SCOUR(M)} = \text{SCOUR(M)} + \text{PCT2(TOTAL1 + TOTAL2 + TOTAL3) (DT)} \quad \text{(B12)}$$

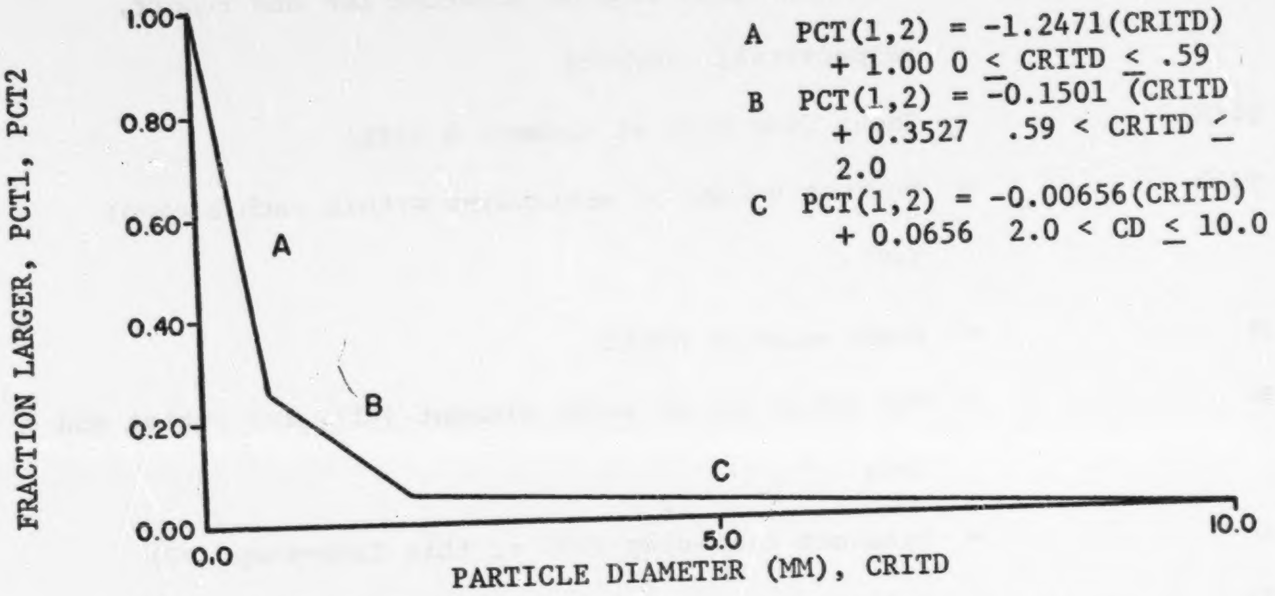


Figure B-1. SIEVE ANALYSIS PLOT FOR SEWER SEDIMENT

$$\text{CPOLL}(M,1,2,2) = [(1 - \text{PCT2})\ (\text{TOTAL1} + \text{TOTAL2} + \text{TOTAL3})\,[/\text{QI}(M) + (1-\text{PCT1})\ \text{SCOUR}(M)/\text{VOL2} \qquad \text{(B13)}$$

$$\text{SCOUR}(M) = (\text{PCT1})\ \text{SCOUR}(M) \qquad \text{(B14)}$$

Step 6. Use Eq. B2 for routing pollutant.

Definition of Variables

SCOUR(M) = Quantity of settled sediment in sewer element M (lb)

CPOLL(M,N,O,IP) = Concentration of pollutant IP at sewer element M (lb/cf)

PCT1, PCT2 = Fraction of sediment with diameter greater than or equal to CRITD

DT = Time increment

TOTAL1 = Sum of all pollutant flow rates from sewer elements immediately upstream (lb/sec)

TOTAL2,TOTAL3 = Pollutant flow rate of incoming DWF and runoff, respectively (lb/sec)

QI(M) = Input flow rate at element M (cfs)

VOL2 = Current volume of wastewater within each element (cf).

M = Sewer element number

N = For inlet end of sewer element (=1), for outlet end (=2)

O = Previous time-step (=1) or this time-step (=2)

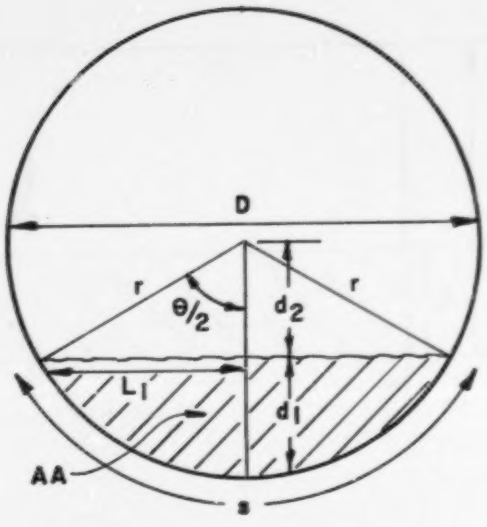
IP = Pollutants: BOD(=1), SS(=2), COLIFORM(=3) and DO(=4)

Hydraulic Radius (Function RADH)

The objective is to compute the hydraulic radius of a given area of flow in a conduit. Let the hydraulic radius of a conduit be defined as:

$$\text{Hydraulic radius} = \frac{\text{Area of flow}}{\text{wetted perimeter}} \qquad \text{(B15)}$$

Examples of the procedure used in calculating the hydraulic radius of a circular conduit and a rectangular conduit are given in Figures B-2 and B-3. Similar procedures are used for a modified basket handle, rectangular conduit with triangular bottom and rectangular conduit with round bottom. All other conduit shapes are found by calculating an equivalent circular diameter and using the procedure given in Figure B-2.



where AA = Area of flow
D = Conduit diameter
r = D/2
$d_1$ = Depth of flow
s = Wetted perimeter
RADH = Hydraulic radius

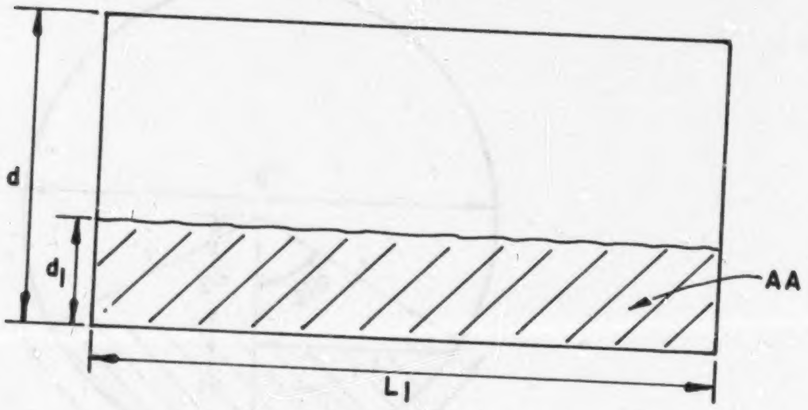
1) For conduit flows less than half full:
   a) $d_2 = r - d_1$
   b) $L_1 = \sqrt{r^2 - d_2^{\,2}}$
   c) $\theta/2 = \tan^{-1} (L_1/d_2)$
   d) $s = r\theta$

2) For conduit flowing over half full:
   a) $d_2 = d_1 - r$
   b),c) Same as in 1
   d) $s = 2\Pi r - r\theta$
   $= r(2\Pi - \theta)$

3) For conduit flowing half full:
   $s = \Pi r$

4) After wetted perimeter has been calculated, RADH = AA/s, Eq. B15.

Figure B-2. CIRCULAR CHANNEL SECTION



where AA = Area of flow
d = Conduit height
$L_1$ = Conduit width
$d_1$ = Depth of flow
s = Wetted perimeter

1) s = $L_1 + 2\, d_1$

2) RADH = AA/s, Eq. B15.

Figure B-3. RECTANGULAR CHANNEL SECTION

1 *Accession Number*

2 *Subject Field & Group*
013B

**SELECTED WATER RESOURCES ABSTRACTS**
**INPUT TRANSACTION FORM**

5 *Organization*
Metcalf & Eddy, Inc., Palo Alto, California
Florida University, Gainesville, Dept. of Environmental Engineering
Water Resources Engineers, Inc., Walnut Creek, California

6 *Title*
STORM WATER MANAGEMENT MODEL

10 *Author(s)*
Lager, John A.,
Pyatt, Edwin E., and
Shubinski, Robert P.

16 *Project Designation*
EPA Contract Nos. 14-12-501, 502, 503

21 *Note*
Set of four volumes: Volume I - Final Report, Volume II - Verification and Testing, Volume III - User's Manual, Volume IV - Program Listing

22 *Citation*

23 *Descriptors (Starred First)*
Water Quality Control*, Computer Model*, Storm Water*, Simulation Analysis, Rainfall-Runoff Relationships, Sewerage, Storage, Waste Water Treatment, Cost Benefit Analysis

25 *Identifiers (Starred First)*
Combined Sewer Overflows*, Urban Runoff

27 *Abstract*

A comprehensive mathematical model, capable of representing urban storm water runoff, has been developed to assist administrators and engineers in the planning, evaluation, and management of overflow abatement alternatives. Hydrographs and pollutographs (time varying quality concentrations or mass values) were generated for real storm events and systems from points of origin in real time sequence to points of disposal (including travel in receiving waters) with user options for intermediate storage and/or treatment facilities. Both combined and separate sewerage systems may be evaluated. Internal cost routines and receiving water quality output assisted in direct cost-benefit analysis of alternate programs of water quality enhancement. Demonstration and verification runs on selected catchments, varying in size from 180 to 5,400 acres, in four U.S. cities (approximately 20 storm events, total) were used to test and debug the model. The amount of pollutants released varied significantly with the real time occurrence, runoff intensity duration, pre-storm history, land use, and maintenance. Storage-treatment combinations offered best cost-effectiveness ratios. A user's manual and complete program listing were prepared.

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